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AIRPHOTO INTERPRETATION OF ENGINEERING SOILS OF KOSCIUSKO COUNTY, INDIANA

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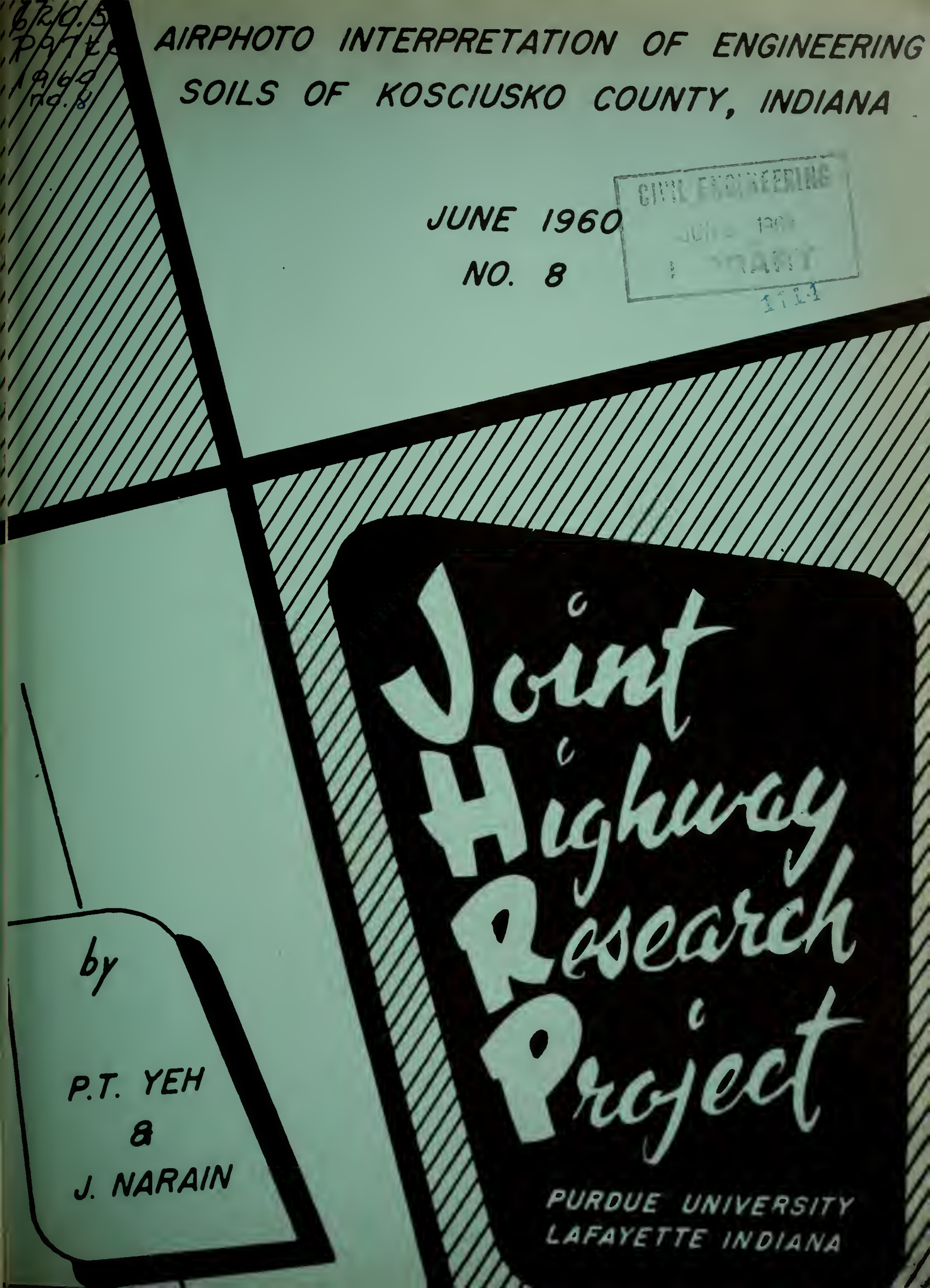

by

P.T. YEH
&
J. NARAIN

Joint
Highway
Research
Project

PURDUE UNIVERSITY
LAFAYETTE INDIANA

Final Report

AIRPHOTO INTERPRETATION OF ENGINEERING SOILS OF
KOSCIUSKO COUNTY, INDIANA

TO: K. B. Woods, Director
Joint Highway Research Project

June 3, 1960

File: 1-5-2B-32

FROM: H. L. Michael, Assistant Director
Joint Highway Research Project

Project No.: C-36-51B

The attached report entitled "Airphoto Interpretation of Engineering Soils of Kosciusko County, Indiana," completes a portion of the project concerned with Engineering Soils mapping from aerial photographs. The report was prepared by P. T. Yeh, Research Engineer and Jag. Narain, former research assistant, Joint Highway Research Project.

The soils mapping of Kosciusko County was done primarily by airphoto interpretation. However, the soil borders are justified by field investigation. To increase the value of the county soil maps, the major soil types were sampled and tests were performed in the soil laboratory. The soil testing data included grain-size analysis, plastic limit, liquid limit, optimum moisture content for maximum dry weight from standard proctor test and CBR. The soils were classified under the Unified Soil Classification System and the Bureau of Public Roads System.

An ozalid print of the engineering soils map and the test data and the appropriate classification listed in a table on separate sheets are included in the back of the report.

Respectfully submitted,

H. L. Michael

H. L. Michael, Secretary

HLM:cw

Attachment

cc: F. L. Ashbaucher
J. R. Cooper
W. L. Dolch
W. H. Goetz
F. F. Havey
G. A. Leonards
J. F. McLaughlin
R. D. Miles
R. E. Mills
C. E. Vogelgesang
J. L. Waling
J. E. Wilson
E. J. Yoder
G. A. Hawkins (M. B. Scott)

AIRPHOTO INTERPRETATION OF ENGINEERING SOILS

OF

KOSCIUSKO COUNTY, INDIANA

by

P. T. Yeh
Research Engineer

and

Jag. Narain
Research Assistant

Joint Highway Research Project
File: 1-5-2B-32
Project: C-36-51B

Purdue University
Lafayette, Indiana

June 8, 1960

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AIRPHOTO INTERPRETATION OF ENGINEERING SOILS
OF
KOSCIUSKO COUNTY, INDIANA

INTRODUCTION

The mapping of the engineering soils of Kosciusko County was accomplished using 9" x 9" aerial photographs having an approximate scale of 1:20,000. These airphotos were taken during July of 1951 in connection with the mapping program of the United States Department of Agriculture. Prints were purchased from the Commodity Stabilization Service, Performance and Aerial Photography Division, United States Department of Agriculture.

Photo interpretation of the land forms and soil textures of the county was accomplished in accordance with accepted principles of observation and inference (1). Field trips were made to the area for the ambiguous details, correlating airphoto patterns with soil textures, and to secure soil samples.

Standard mapping symbols developed by the staff of the Airphoto Interpretation Laboratory, School of Civil Engineering, Purdue University, were employed to delineate land forms and soil textures. The text of this report largely represents an effort to overcome the limitations imposed by adherence to a standard symbolism.

An approach towards better utilization of Engineering Soils Maps of Indiana has been made with the inclusion of laboratory classification and test data for the principal soils represented within this county. Certain soil types, of obviously inferior engineering qualities, were

not sampled but were indicated on the soils map. These soils include principally organic soils such as muck, peat and highly organic soils which are of only limited extent.

Glacial soil deposits are not uniform in composition throughout a large area. Therefore, the soil samples obtained and the test data reported are valuable when comparing one soil with another. Detailed field sampling is necessary for final design purposes. Additional engineering soil test data and profiles may be obtained from "The Formation, Distribution and Engineering Characteristics of Soils" (2).

Liberal reference was made to the Soil Survey of Kosciusko County, Indiana prepared by the Agricultural Experiment Station, Purdue University and the U. S. Department of Agriculture in 1927 (3). In many instances the agricultural soils map did provide a convenient endorsement of the photo interpretator's judgment.

INTERPRETATION OF AREA

General

Kosciusko County is situated in the northern part of Indiana as noted on Figure 1. The outline of the county is approximately a quadrangle. It has a north-south measurement of 27 miles and a maximum east-west stretch of 22 miles. The area of the county is about 541 square miles (3). Warsaw the principle city and the county seat is centrally located. It is served by Federal highway No. 30, and state routes No.'s 15 and 25. Two railroads also serve the community. A population of 33,002 inhabitants resided within the county with 6625 reported for Warsaw alone in the 1950 census (4).

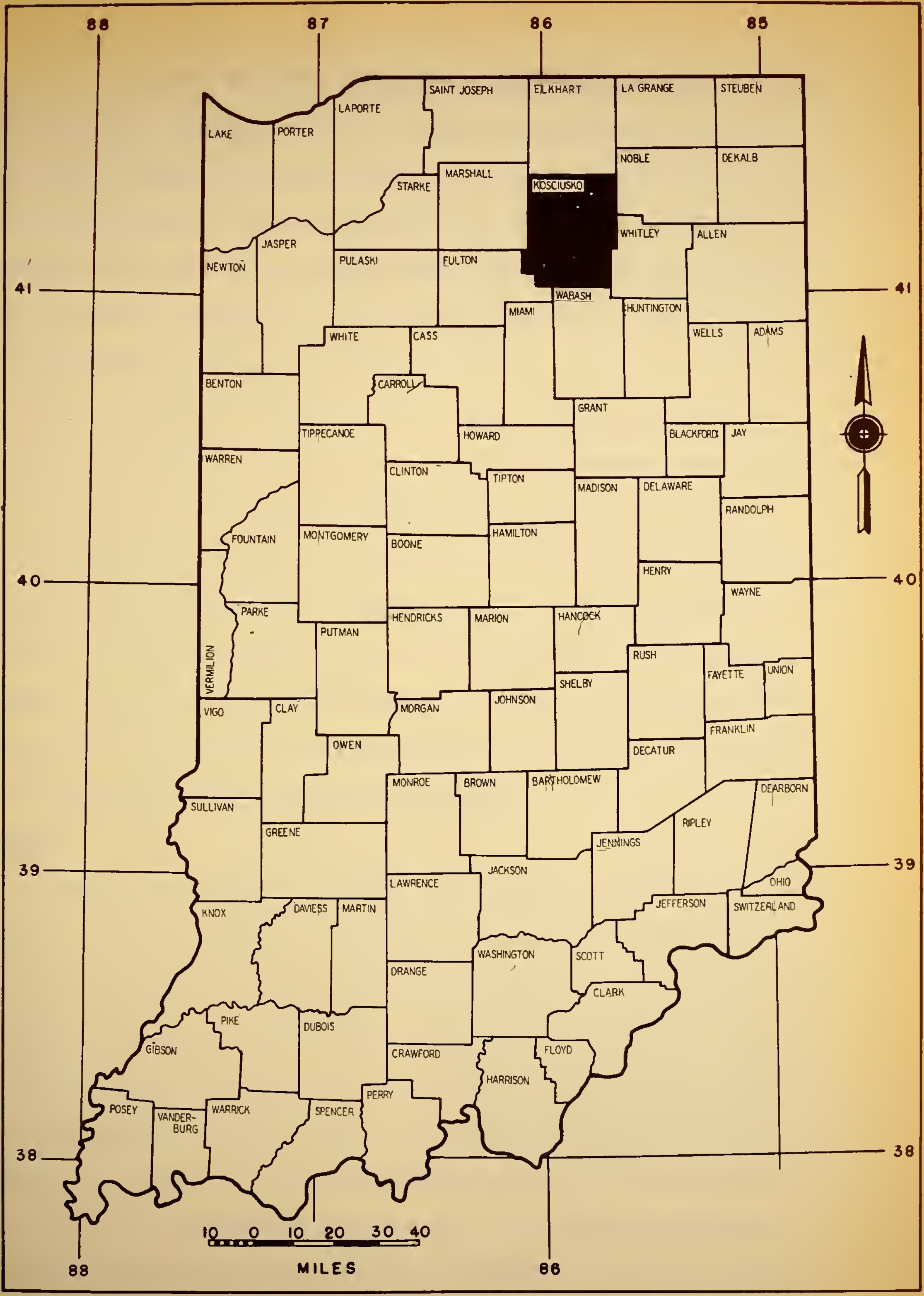


FIG. I LOCATION MAP OF KOSCIUSKO COUNTY

Land use within Kosciusko County is principally devoted to farming with 91.6% of the total area reported under cultivation in 1954 (5). Much of the uncultivated portion including wooded lands and slopes unsuited for cultivation lies along the swampy alluvial plain of the Tippecanoe River and around the large lakes (see Figure 2). Occasional woodlands remain generally in areas characterized by high water table and water-logged soil conditions.

There are numerous basins and depressions within the county, many contain lakes and muck areas ranging in size up to several square miles. Many famous resort lakes such as Lake Winona, Chapman Lake, Webster Lake, Tippecanoe Lake, Lake Syracuse and Lake Wawasee are scattered in the northeastern quarter of the county. Among them, Lake Wawasee, located on the northeastern corner of the county is the largest lake of Indiana. Due to the aggressive program of artifical drainage the water table on all flat lands has become permanently lower. Many small lakes have disappeared since the county was settled.

Climate

Kosciusko County lies within the humid, middle altitude continental climatic region. The polar continental air masses meet those from a high-pressure region in the mid-Atlantic area and create highly variable weather condition. The mean summer temperature is 73.3 degrees, ranging from a maximum of 110 degrees to a minimum of 37 degrees. The mean winter temperature is 28.6 degrees, with a minimum of -23 degrees to a maximum of 69 degrees. The annual precipitation is 36.64 inches, about half of it falls between May and September. The normal monthly and annual temperature and precipitation as well as the extremes at Warsaw are given in table I (6).

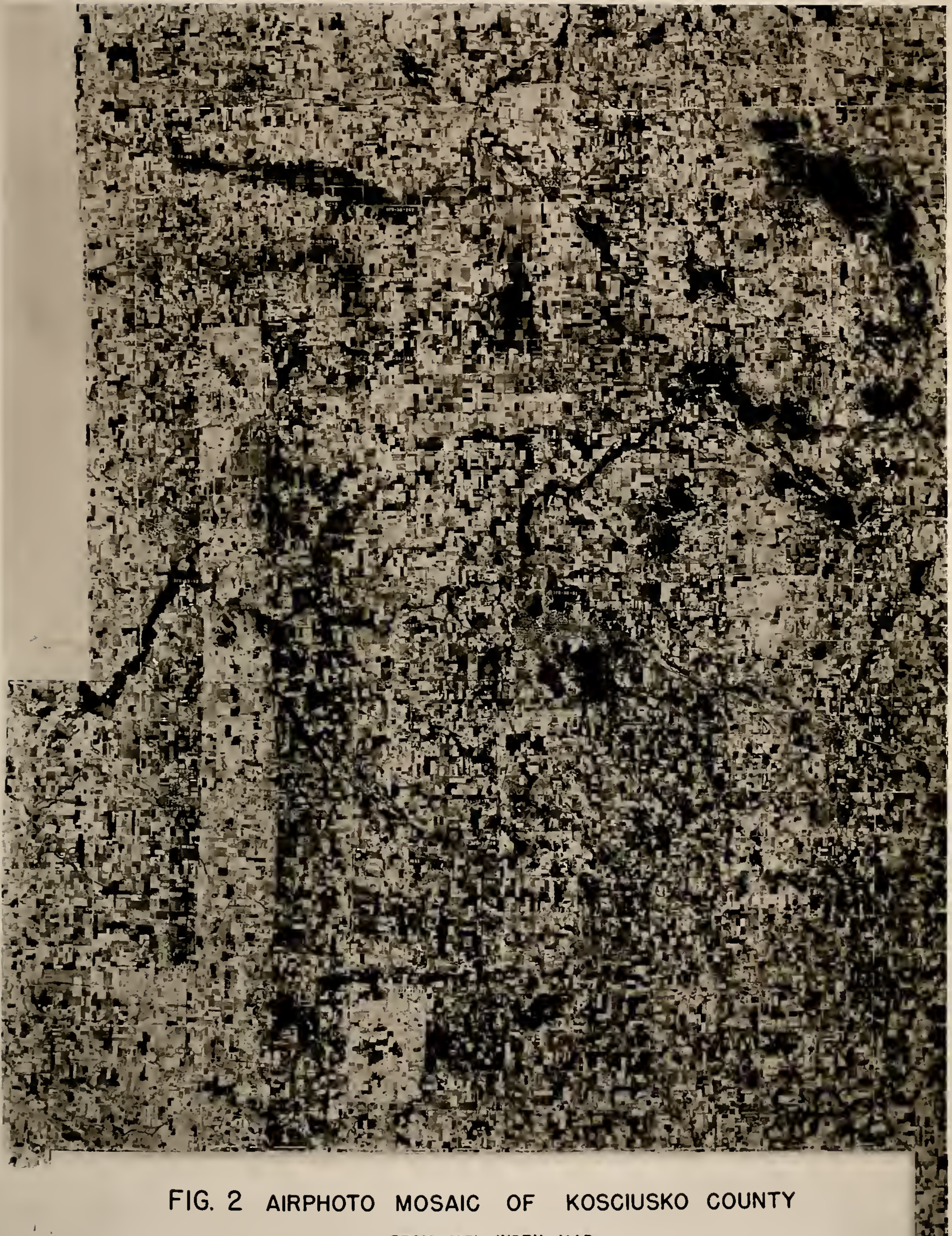


FIG. 2 AIRPHOTO MOSAIC OF KOSCIUSKO COUNTY

FROM 1951 INDEX MAP

TABLE I

Normal monthly and annual temperature and precipitation at Warsaw, Kosciusko County, Indiana.

Month	Temperature			Precipitation		
	Mean °F	Absolute Maximum °F	Absolute Minimum °F	Mean Inches	Total Driest Year Inches	Total Wettest Year Inches
January	27.7	69	-20	2.12	1.87	1.92
February	28.5	68	-18	1.92	1.26	2.63
March	37.2	85	- 6	3.04	2.43	5.22
April	49.2	100	16	3.56	4.14	5.84
May	60.5	98	26	3.85	2.81	2.72
June	71.1	104	37	4.21	2.64	5.31
July	75.5	110	43	3.46	1.59	4.82
August	73.3	106	37	3.32	2.93	4.74 (Estimate)
September	65.9	103	28	3.28	1.71	1.09
October	54.8	92	20	3.41	0.95	12.67
November	40.4	85	- 4	2.53	0.97	1.50
December	29.5	66	-23	1.94	1.09	2.10
Year	51.1			36.64	24.39 (in 1953)	50.29 (Estimate) (in 1954)

The data are compiled from records of Winona Lake Station (Elevation 865 ft.) during the period of 1909 to 1946 and from the Warsaw Waterwork (Elevation 808) from 1946 to Jan. 1960.

Physiography

Kosciusko County lies in the Steuben Morainal Lake Section of the Norther Moraine and Lake Physiographic Region of the state, which is a part of the Eastern Lake Section of the Central Lowland Province (7).

Topography

The surface features of Kosciusko County vary from level plains to undulating and rolling country. The general elevation varies from 800 to 950 feet above sea level. The maximum elevation of 1025 feet is attained by a hill located about four tenths of a mile north of Dewart Lake on the southwestern corner of Sec. 19 T34N R7E (8). The lowest altitude is about 770 feet (7). Basin and depression are numerous. Many are lake and muck areas; some are merely sags in the surface.

The northwestern quarter of the county is a slightly uneven plain with a few prominent ridges which are only a few feet above the surrounding flats.

The northeastern quarter of the county is more diversified. Much of it consists of uplands of pronounced elevation. The configuration of the ridges ranges from gently rolling to pitted with kettle depressions. The prominent lakes of the county are located chiefly in this quarter. Most of them are surrounded by terrace-like outwash plains which are broad, slightly undulating areas usually bordered on the side toward the water by sharp declines of from 10 to 40 feet.

The southern half of the county consists chiefly of uplands of moderate elevation and mild relief. There are a few lakes, but numerous mucky shallow depressions occur. Rolling lands are of limited extent and are usually found near the lakes and marshes.

Drainage Features

The principal stream in Kosciusko County is the Tippecanoe River. It drains the central part of the county westerly into Marshall County as shown in Figure 3. A string of lakes are connected by this sluggish river on the eastern part of the county.

The northeastern and north central part of the county is drained by Turkey Creek Ditch in a westerly direction before it enters Elkhart County to join the Elkhart River to the north. Lake Wawasee is drained by this system.

Areas (about 48 square miles) on the northwestern corner of the county are drained in a westerly direction by Middle Fork Yellow River and Dausma Ditch into Marshall County. Both belong to the Kankakee drainage system.

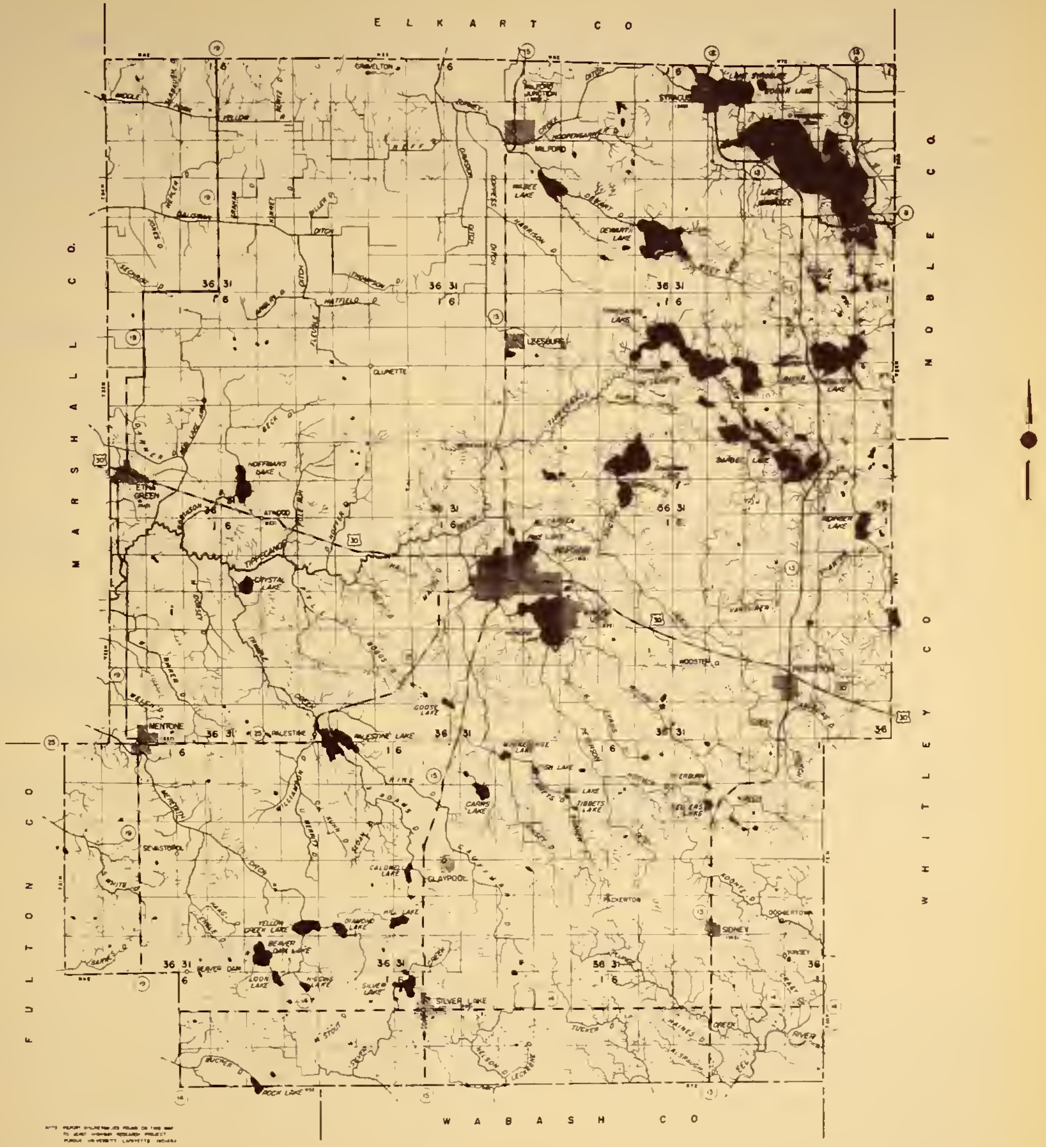
About 52 square miles of area located in the southeastern corner of the county is drained by the Eel River System. Eel River crosses the southeast corner of the county in a distance of less than four miles.

The rectilinear drainage pattern shown on the drainage map (Figure 3) indicates some streams have been dredged and many ditches added to facilitate the surface drainage as well as to improve the high ground water situation.

On the sandy outwash plains north of Warsaw, little or no surface drainage lines have been developed.

Geology

Kosciusko County was glaciated. The glacial covering attains a thickness of more than 300 feet and the underlying bedrock are nowhere exposed (9). Logan reports that the bedrocks of the county are mainly of Devonian age. A small area in the southern part of the county may



DRAINAGE MAP
KOSCIUSKO COUNTY
INDIANA
PREPARED FROM
1951 AAA AERIAL PHOTOGRAPHS
BY
JOINT HIGHWAY RESEARCH PROJECT
AT
PURDUE UNIVERSITY
1956

SCALE OF MILES

POLYCONIC PROJECTION

FIG. 3 DRAINAGE MAP OF KOSCIUSKO COUNTY

have Silurian strata underlying the drift and a small area in the northern part of the county may have rocks of Mississippian age under the drift.

The following geological divisions have been recognized in the record of wells.

Quaternary - - - Clays, sands, gravels (recent)
Clays, sands and gravels (Pleistocene)

Mississippian - - - Shales, sandstone, (Borden)

Devonian - - - Shales (New Albany)
Limestone (Sellersburg, Jeffersonville)
Sandstone (Pendleton, Schoharie)

Silurian - - - Limestones (Huntington, Liston Creek)
Shale (Mississinewa)

Ordovician - - - Limestones, shales (Richmond, Maysville, Eden)
Limestone (Trenton, Mohawkian)

AIRPHOTO INTERPRETATION OF SOIL AREAS

The soils of Kosciusko County include deposits of glacial, glaciofluvial, lacustrine and alluvial origins. Some of the glaciofluvial materials have been modified by wind, so that eolian features superimposed over material of other origins create somewhat complex soil patterns.

GLACIAL DEPOSITED MATERIALS

The entire county was covered by continental glaciers, therefore, the surface soils of Kosciusko County are chiefly developed on glacial drift of Wisconsion age. The various deposits are discussed as follows:

Moraines

A study of moraines of northern Indiana and southern Michigan by Leverett (10) concluded that the portion of the ice sheet from the Saginaw basin which moved southwestward into Indiana melted back and disappeared from northern Indiana while the Lake Michigan and Huron-Erie lobes still extended into the state. This convergence of the lobes resulted in a coalescence more or less complete and maintained until destroyed by a rapid recession of the Saginaw Lobe. This phenomenon accounts for the development of the moraines in Kosciusko County.

Moraines represented within Kosciusko County include the Packerton, the Burket, the Breman and the New Paris moraines.

(1) Packerton Moraine

The name of the Packerton moraine is obtained from the village of Packerton in southeastern Kosciusko County. This is an interlobate moraine formed by the eastern limb of the Saginaw morainic lobe and the northern limb of the Erie morainic lobe (see Figure 4). The crowding of these two ice-lobes resulted in great masses of morainal material.

The Packerton moraine covers much of the southeastern third of the county; this moraine traverses the county in a southwest-northeast direction. The moraine generally has rolling topography within the county. The swells and hills are 10 to 60 feet in height. Occasionally, however, less rolling or undulating tracts and hilly areas may be encountered. The hilly areas are usually found near the lakes and marshes.

The moraine forms the divide between the Tippecanoe and Eel Rivers. The crest of the moraine can be traced from the divide of

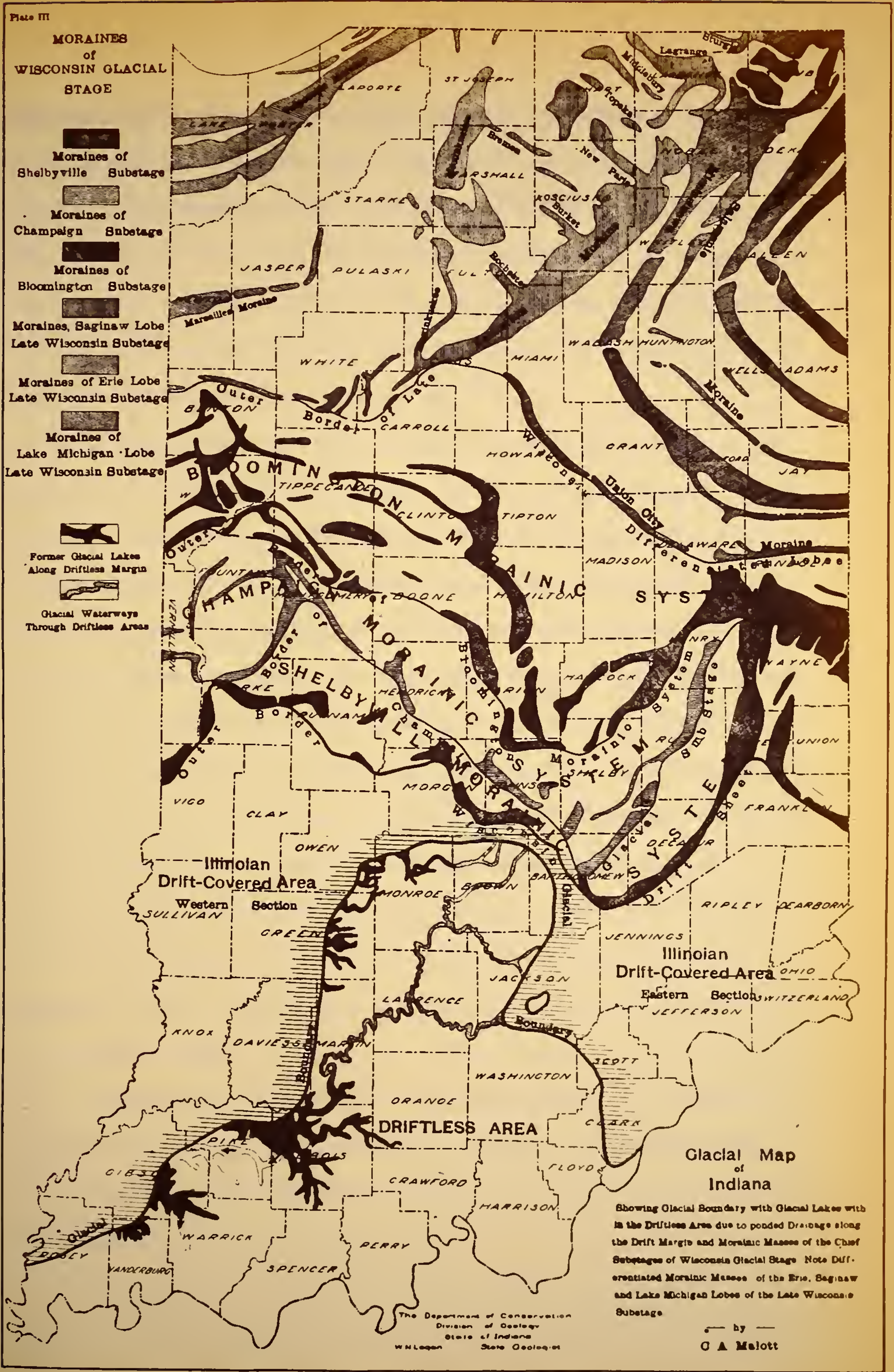


Plate III
MORAINES of WISCONSIN GLACIAL STAGE
Moraines of Shelbyville Substage
Moraines of Champaign Substage
Moraines of Bloomington Substage
Moraines, Saginaw Lobe Late Wisconsin Substage
Moraines of Erie Lobe Late Wisconsin Substage
Moraines of Lake Michigan Lobe Late Wisconsin Substage
Former Glacial Lakes Along Driftless Margin
Glacial Waterways Through Driftless Areas
Marseilles Moraine
Outer Border of Late Wisconsin
BLOOMINGTON MORAINIC SYSTEM
CHAMPAIGN MORAINIC SYSTEM
SHELBYVILLE MORAINIC SYSTEM
Outer Border of Champaign
Outer Border
Union City Moraine
Wabash Moraine
Mississinewa Moraine
Differentiated Moraine Lobes
Illinoian Drift-Covered Area Western Section
Illinoian Drift-Covered Area Eastern Section
Glacial Boundary
DRIFTLESS AREA
Lagrange
Sturgis
Middlebury
Topeka
Bremen
New Paris
Burket
Rochester
LAPORTE
ST JOSEPH
STARKE
MARSHALL
KOSCIUSKO
JASPER
PULASKI
FULTON
WHITE
CARROLL
MIAMI
WABASH
HUNTINGTON
WELLS
ADAMS
ALLEN
GRANT
HOWARD
JAY
TIPPECANOE
CLINTON
TIPTON
MADISON
DELAWARE
BENTON
FOUNTAIN
BOONE
HAMILTON
HANCOCK
HENDRICKS
WAYNE
UNION
VERMILLION
PARKE
PUTNAM
MORGAN
SHELBY
FRANKLIN
DECATUR
RIPLEY
DEARBORN
VIGO
CLAY
OWEN
MONROE
BROWN
BARTHOLOMEW
SULLIVAN
GREENE
JENNINGS
OHIO
SWITZERLAND
JEFFERSON
JACKSON
LAWRENCE
KNOX
DAVIESS
MARTIN
SCOTT
WASHINGTON
ORANGE
CLARK
PIKE
DUBOIS
GIBSON
CRAWFORD
FLOYD
HARRISON
POSEY
WARRICK
PERRY
VANDERBURG
SPENCER
Glacial Map of Indiana
Showing Glacial Boundary with Glacial Lakes with in the Driftless Area due to ponded Drainage along the Drift Margin and Morainic Masses of the Chief Substages of Wisconsin Glacial Stage Note Differentiated Morainic Masses of the Erie, Saginaw and Lake Michigan Lobes of the Late Wisconsin Substage
by C A Malott
The Department of Conservation Division of Geology State of Indiana W N Logan State Geologist

FIG. 4 GLACIAL MAP OF INDIANA

the drainage basins on the drainage map as, for example, at Packerton (Figure 3). This area has a haphazard drainage pattern, especially on the northwestern side of the moraine where basins are numerous. On the southeastern part of the moraine fewer basins are found and more surface drainage systems have been developed. The main streams on the southeastern part show a certain degree of parallism indicating the uniform slope of the moraine.

As indicated on the engineering soils map, the moraine is composed of soils of varying texture. On the northwestern part of the moraine, muck kettles or depressions as well as small lakes are numerous and the widely spaced surface drainage systems as shown on the drainage map (Figure 3) indicate the pervious nature of the coarse-textured deposit. The south half of the moraine is composed of more impervious and clayey deposits. The Saginaw lobe carried granular loads from the north while the Erie lobe brought in chiefly clayey deposits; therefore, the difference in texture of this interlobate moraine can be expected. According to the geological literature (7) the Saginaw deposit may subsequently have been partially removed or over-ridden by the Erie lobe; hence, the granular surface deposit from the Saginaw loba may have been modified to a certain extent.

The sandy textured deposits are chiefly Miami sandy loam with small areas of Miami loam, Brookston silt loam and Coloma sand. A limited extent of Bellefontaine soils may be found near some kettles or lakes especially the larger lakes.

The typical soil sample was taken at site No. 23 located about one mile east of Claypool. The surface soils or topsoils (A-horizon)

are about 12 inches in thickness. It is a loose sandy soil with little organic matter. Small stones and gravel are usually present. The B-horizon taken between 12 and 24 inches from the surface contains 5.0% gravel, 64.7% sand and 30.3% fines. It has a liquid limit (LL) of 13.6% but is non-plastic. The C-horizon taken from 2 to 3 feet below the surface reveals 5.7% gravel, 24.4% sand and 69.9% fines. The LL indicates an increase to 21.3% and a plastic index (PI) of 7.7%. The silty sand B-horizon has a stronger supporting power than the till C-horizon. It shows a CBR value of 15.0 while the C-horizon shows only 3.7. The subsoil (B-horizon) is classified as SM and A-2-4 by the USCS (Unified Soil Classification System) and BPR (Bureau of Public Roads) classification respectively. CL or A-4 (7) soil is the classification of the parent material of the area.

Another soil sample taken at site No. 24 about three and a half miles directly south of the previous site shows essentially the same parent material. However, the B-horizon is much less sandy than that of site No. 23. It has 3.9% gravel, 43.7% sand and 52.4% fines. The LL and PI are 25.8% and 8.8% respectively. It is classified as CL or A-4 (3) soil. Since this test site is close to the silty-clay morainic border or the crest of the Packerton moraine less granular subsoil can be expected.

Test site No. 3 is located west of Claypool in a depression. The water table was about 2 feet from the surface during the summer months. The B-horizon sample was taken at 3 feet from the surface. The dark colored soil shows 8.2% of gravel, 37.5% of sand and 54.3% of fines. It has a LL of 28.9% and a PI of 11.9% and is

classified as CL or A-6 (4) soil. The parent material sampled at 5 feet from the ground surface indicates more sand and less plasticity. It is believed to be the same as those at site No. 23.

Coarser materials may be found close to the northern border of the moraine especially adjacent to the outwash plain. Test site No. 21 located about four miles southeast of Warsaw illustrates this condition. The dirty gravelly B-horizon obtained between 1 to 3 feet from the surface contains 15.4% of gravel, 63.3% sand and 21.3% fines. It has a LL of 15.9% and is non-plastic. The parent material between 3 to 4 feet from the ground surface reveals a slight increase of gravel (21.8%) decrease of sand (47.6%) and increase of fines (30.6%). The soil is not plastic. The bearing capacity of the B-horizon is greater than that of the parent material (38.0 vs 22.5). Both soils are classified as SM or A-2-4 soils.

On the southeastern slope of the moraine less granular soil can be expected. Soils are chiefly Miami silt loam with small areas of the Crosby and Brookston series. Exception is found in an area about six square miles located northeast of Sidney. The soil of this area is Miami sandy soil, same as those on the northern slope.

A representative soil sample of the clayey moraine was obtained at site No. 22 about two miles northeast of Packerton. The surface soil consists of about 8 inches of friable sandy silt. Stiff clayey till with very little gravel lies below the surface soil. The B-horizon taken from 1 to 3 feet below the ground surface consists of 2.7% gravel, 19.2% sand, and 78.1% fines. It has a LL of 40.2% and PI of 21.6%. This clayey horizon is very weak in supporting power. It has a CBR value of 1.2 and is classified as CL or A-7-6 (14)

soil. The parent material taken below 36 inches indicates less plasticity than the B-horizon. It is composed of 2.8% gravel, 29.0% sand and 68.2% fines. Both the LL and PI reduced to 24.6% and 11.0% respectively. Higher bearing capacity (CBR = 8.6) is also shown in this horizon. The parent material is classified as CL of A-6 (7) soil by the USCS and the BPR classification respectively.

From the distribution of the gravel pits as shown on the soils map the clayey southeastern slope of the Packerton moraine shows only one small pit located at Kinsey close to the southeastern border adjoining a muck deposit. On the northern slope of the moraine more gravel pits are located. Disregarding the fact that most of the gravel supplies of the county come from the outwash plains, the distribution of gravel pits within the Packerton moraine gives good indication of the difference in characteristic between the north and south sections as indicated on the engineering soils map.

(2) Burket moraine

The Burket moraine is an off-shoot of the Packerton moraine (see Figure 4). It is located just north of Burket running in a northwest to southeast direction. The moraine is separated into two parts by the terraces along Trimble Creek and a ground moraine east of Palestine. Hilly topography is predominant on the Burket moraine except in the portion west of Palestine where rolling topography is found.

Natural surface drainage is poorly developed on the moraine (see Figure 3). However, man-made ditches are common in this area

for the lowering of the ground water table of the depressions, and muck channels. Muck kettles are less in number than on the northwestern slope of the Packerton moraine but are considerably more developed on the ridge moraine than on the surrounding ground moraine (till plain) area. A few gravel pits are found on the northern half of the moraine and are a good indication of the granular nature of the moraine.

The soil of this morainic belt is mainly Miami sandy soil with small areas of Bellefontaine soil adjoining and Coloma sand scattered northeast of Palestine Lake. In the depressions, Brookston soil and Clyde soil are dominate. The general soil found in this area is believed to be similar to the soil at site No. 23.

(3) Breman Moraine

The Breman moraine is one of the several moraines formed during the rapid recession of the Saginaw lobe (see Figure 4). It is a weak moraine especially on its eastern part in Kosciusko County. The moraine has its strongest development in the vicinity of Breman of Marshall County where it raises 20 to 25 feet above the marshy plain to the north and the undulating plain to the south. It enters Kosciusko County on the northwestern corner in an easterly direction. It is a low ridge of 10 to 15 feet in height above the muck channel to the north and outwash plain on the south. Due to the subdued topography, a surface drainage system is not well developed. A few muck depressions 10 to 20 feet below the general ground elevation are scattered throughout this region.

The soils of this section are principally Crosby and Brookston soils. The typical soil sample was obtained at site No. 10 located

about two miles south and two miles east from the northern and western county borders respectively. The surface soil, about 12 inches deep, is a very friable fine-textured silty soil. Small stones and gravel are usually numerous on the surface. The B-horizon taken 2 to 3 feet from the surface contains 3.3% gravel, 37.5% sand and 59.2% fines. The LL and PI of this subsurface soil is 21.8% and 9.3% respectively. It has a CBR value of 6.4 and is classified as CL and A-4 (5) soil. The parent material taken 3 feet below the ground surface shows essentially the same soil material as the subsoil above. It reveals a slight increase of gravel (7.2%) and a decrease in fines (55.4%). The plasticity of the parent material is a little less than the subsoil. It has a LL of 19.5% and PI of 7.7%. The bearing capacity increases to 16.8. As classification of soil is concerned the B and C horizons fall in the same category - CL and A-4 (5).

There is a high pitted plain located about two miles south of the aforesaid moraine running in a southeasterly direction toward Warsaw. This plain was considered as the continuation of the Breman moraine by Leverett as shown in Figure 4 (9). Judging from the airphoto patterns and the texture of the deposits it appears more likely to be a pitted outwash plain; therefore, it is considered under outwash material in this report.

Two areas about three square miles in extent at or near Atwood are believed to be part of the Breman moraine. The area has a very rugged topography. It rises to 40 feet or more above the surrounding outwash plains. As indicated from the drainage map, only short gullies are developed along the edge of the deposit leading toward

the depressions. The soil in this morainic area is chiefly granular Bellefontaine soil associated with Miami and Coloma soils.

(4) New Paris Moraine

The New Paris moraine traverses Kosciusko County in a northwest-southeasterly direction across the northeastern quarter of the county. The moraine has its most prominent feature at New Paris in Elkhart County. In this county the moraine is somewhat loosely assembled into a series of undulating and rolling tracts.

The most prominent morainic relief is found on the kettle-kame morainic belt located about two miles southeast of Milford and northeast of Wabee Lake and Dewart Lake. This morainic ridge is approximately 50 feet above the outwash plain on the north and more than 80 feet higher than the outwash plain to the south. Within the morainic belt, the topography is hilly with a number of shallow basins. The highest altitude of the county is attained in this morainic belt north of Dewart Lake. Local relief changes of more than 130 feet can be found in this area.

A pitted airphoto pattern and a weakly developed surface drainage pattern indicates the pervious and granular nature of the deposit. A number of gravel pits located within this area further confirm the granular prediction. The soil is principally Bellefontaine soil. The typical soil profile will be similar to sample No. 15 to be discussed later.

The eastern part of the moraine located north of Webster Lake is also very rugged in topographic expression. The hills are about 920 to 950 feet in altitude, and local relief of 60 feet is not uncommon. The pitted photo pattern is less pronounced than in the

area to the west. It appears more sandy in texture than the former one. The soil profile is believed to be similar to those as found in test site No. 23 and No. 21.

A tract of subdued morainic topography lies between the New Paris moraine and Lake Wawasee. Another similar tract is located and mapped north of Tippecanoe Lake. Both areas have undulating topography with occasional rolling lands. Local relief is generally less than 30 feet with a maximum of 45 feet at one point. A well developed surface drainage is found especially on the northern slope south of Lake Wawasee. The soil in this area is chiefly Miami soil with Brookston soil in the depression. A representative soil sample was obtained at site No. 19, about two and a half miles east of Dewart Lake. Below the silty surface soil which was about 12 inches in thickness, the B-horizon taken from 1 to 3 feet consisted of 4.7% of gravel, 45.9% sand and 49.4% of fines. It has a LL of 20.5% and a PI of 6.7%. The CBR value of this soil is 12.0. The subsoil is classified as SM-SC or A-4 (3) soil. The C-horizon taken from 3 to 5 feet below the surface shows more gravelly (17.5% gravel) and less sandy (31.2% sand) than the subsurface soil. The amount of fines and the plasticity property as well as the bearing strength of the soil are essentially the same as those of the B-horizon. Because of the slight increase of fines the soil is classified as ML-CL soil by the USCS, but by the BPR system it remains the same as A-4 (3) soil.

About three miles east of Milford there is a large depression which contains mainly Brookston soil A clay pit is located within this area. The parent material is almost pure clay. A sample taken

below 6 feet from the surface at test site No. 20 reveals the following composition: gravel 1.7%, sand 9.9% and fines 88.4%. It has a LL of 36.0% and PI of 13.7%. This clayey soil is classified as CL or A-6 (12) soil. It is weak in bearing capacity with a CBR value of only 3.1.

North of Lake Syracuse the moraine has an undualting topography. The soil is essentially a silty Miami soil similar to test site No. 19. No gravel pit is located within this silty moraine area.

Ground Moraine

(1) Sandy-Clay Ground Moraine

The most extensive sandy-clay ground moraine or till plain is located in the intermorainic zone south of the Tippecanoe River and northwest of the Packerton moraine. This ground moraine rises gradually from the Tippecanoe River toward the Packerton moraine. It exhibits an undulating topography. Small knolls or ridges of sand or sandy-gravel material may be found. Peat or muck kettles are not as common in this ground moraine zone as they are in the Packerton ridge moraine. Surface drainage in the form of gullies is not well developed, therefore, a considerable number of ditches are used to facilitate the runoff in this area. The drainage density, which is a general indicator of the perviousness or imperviousness of the soil, is better developed in comparison to the northern slope of the Packerton moraine.

Soil in this ground moraine area is principally Miami loam with Brookston loam in the depression. Several sampling sites were selected in this area. Although the subsurface soils show considerable variation, the parent material is similar at all sites.

A soil sample at site No. 6 located near the western border of the county about three miles west of Sevastopol shows the B-horizon at 1 to 2 feet below the silty top soil to contain considerable gravel (19.1%) and sand (37.6%). The soil contains 43.3% of fines and has a LL of 27.3% and a PI of 10.4%. The soil in the B-horizon is classified as SC or A-6 (2) soil. The C-horizon taken between 3 and 4 feet below the surface shows a slight decrease of gravel (14.4%) an increase of sand (46.1%). The soil exhibits a LL of 22.8% and a PI of 9.9%. It is considered as SC or A-4 (3) soil. The bearing capacity of the subsoil (CBR = 10.9) is higher than that of the parent material (CBR = 5.4) of this site.

Another site (No. 5) is located near Beaver Dam at the southwestern corner of the county. The soil is more clayey than that of site No. 6. The B-horizon taken between 1 to 2 feet from the surface has 6.0% of gravel, 27.3% sand and 66.7% of fines (LL of 34.6% and PI of 17.1%). It is classified as CL or A-6 (9) soil. The C-horizon sampled at 4 feet below the surface is composed of 9.2% gravel, 33.0% sand and 57.8% fines. The soil taken in this horizon is much less plastic (LL = 22.8%, PI = 9.9%) than the B-horizon. It is classified as CL or A-4 (5) soil. Both the B and the C horizons show about the same bearing strength in the CBR tests (8.3 and 8.0 respectively).

As mentioned previously patches of sand and gravel are found in this region. Site No. 2 located about two and a half miles west of Claypool is used for illustration. The surface soil to about 2 feet is sandy. The B-horizon taken at 3 feet below the surface shows 3.2% of gravel, 58.5% of sand and 38.3% of fines.

It has a LL of 20.6% and PI of 7.7%. It is classified as SC or A-4 (1) soil.

Eight feet of sand is found at test site No. 1 situated on the bank of a channel about two miles southeast of Mentone. The parent material taken below the sand shows 7.7% of gravel, 40.5% of sand and 51.8% of fines. With a LL of 21.2% and PI of 8.7% the soils are classified as CL or A-4 (3) soil.

(2) Sandy-Silt Ground Moraine

The sandy-silt ground moraines are located in the northwestern quarter of the county. The largest unit extends two to six miles southwest of Milford. Others are situated north of Atwood and north of Etna Green in the west central part of the county.

The sandy-silt ground moraines have undulating to flat topography. From the drainage map in this report (Figure 3) it is obvious that surface drainage systems are poorly developed in these areas. The airphoto shows a mottled photo tone, typical of ground moraines of sandy-silt textures.

The soil of these areas mainly Crosby, Brookston and Clyde soils. Soil sample No. 13 obtained on the ground moraine west of Milford shows many sand lenses in the soil profile.

The B-horizon taken 2 to 3 feet below the surface is quite sandy in texture. It contains 3.8% gravel, 58.6% sand and 37.6% fines. This SC or A-4 (1) soil has a LL of 22.2% and PI of 10.3%. The C-horizon taken 4 to 5 feet from the surface shows a mixture of gravel (4.4%), sand (46.2%) and fines (49.4%) with a LL of 16.4% and PI of 3.7%. The soil is classified as SM or A-4 (3) soil.

Both the subsoil as well as the parent soil have nearly the same CBR values (6.3 and 7.0 respectively).

(3) Clayey Ground Moraine

North of the vast muck channel located on the northwestern corner of Kosciusko County there lies a clayey ground moraine. This is a very smooth till plain with scarcely a noticeable knoll in the area. The entire ground moraine is void of muck pockets. The clayey texture of the soil is reflected from the surface drainage pattern as shown in Figure 3. The dendritic pattern on the north side of the Middle Fork Yellow River is strikingly different from the area to the south.

Crosby and Brookston are the major soil series in this area. Soil samples taken at site No. 11 and site No. 12 indicates the uniformity of the soil in this area. The B-horizon taken from site No. 11 at a depth of 2 to 3 feet shows 5.4% of gravel, 35.5% of sand and 59.1% of fines. It has a LL of 25.5% and PI of 11.9% and is classified as CL or A-6 (6) soil. The C-horizon taken at the same site at 3 to 4 feet below the ground surface reveals an increase of fines (65.5%) and plasticity (LL = 29.6%, PI = 15.0%) and a slight decrease of gravel (3.9%) and sand (30.6%). The parent soil is classified as CL or A-6 (8) soil.

The C-horixon taken 4 feet from the ground surface at site No. 12 located about four miles east of site No. 11 shows about the same character as at site No. 11. However, grain-size distribution reveals the soil containing more fines (78.8%) and a decrease of bearing power exhibited from the test accordingly (CBR = 3.7 vs

CBR of 6.7 at site No. 11). The parent soil falls in the CL or A-6 (11) soil category.

Eskers and Kames

A few eskers and kames exist within Kosciusko County. The eskers are short and small in size and may be ice contact deposits or crevasse fillings. In form they are quite similar to the sand dune formation on the pitted outwash plain northeast of Atwood. One of the eskers is located just west of Dewart Lake. The kames are much more easily delineated from the airphotos. An outstanding one lies just north of Atwood. It has been excavated as a gravel pit. Another is located on the sandy ground moraine about two miles west of Milford.

The granular texture of these deposits is confirmed by the presence of gravel pits. The soil developed on these formations are generally of the Bellefontaine series.

WATER DEPOSITED MATERIALS

Owing to the enormous amount of melt-water flowing across Kosciusko County during the late glacial stage, vast amounts of material were deposited or redeposited along the major channels and upon the glacial-fluvial plains. The various water deposited materials are discussed as follows.

Outwash Plains

About one quarter of the county is composed of outwash plain deposits. A major outwash plain complex is located in the north central part of the county. The outwash deposits vary considerably in texture.

(1) Gravelly Outwash Plain

The gravelly outwash plain has a distinct pitted appearance. It may be called a pitted outwash plain. The most outstanding one

lies northwest of Warsaw. The plain is about 10 to 20 feet higher than the surrounding sandy-gravelly plains. Swales of 10 to 20 feet in depth are common in this plain. At its eastern end in the bend of the Tippecanoe River north of Warsaw a series of sharp knolls, 30 to 50 feet in height, are found. The plain was mapped as Breman moraine by Leverett (9). The overall airphoto patterns and the topography suggest this area is more appropriate to be considered as pitted outwash plain. Pitted infiltration marks are numerous and current markings are also clearly shown. The topography is gentle on the wester portion but appears to be more rugged toward the higher altitude in the east. A few sand dunes are recognized from airphotos. Large muck kettles are found on the western portion of the plain. Except for a few short gullies developed around the basins, the entire area is void of surface drainage (see Figure 3). The granular nature of this deposit is further confirmed by the number of gravel pits located on this plain. The soil in this pitted outwash plain belongs to the Bellefontaine series.

The sample taken at site No. 15 located about seven miles west of Leesburg shows below the loose, coarse-sandy surface soil a somewhat finer, sandy subsurface soil. The B_1-horizon taken between 1 to 2 feet from the ground surface has 19.4% of gravel, 41.8% of sand and 38.8% of fines. This layer shows considerable plasticity (LL = 30.0% and PI = 14.0%). The CBR is 12.5. It is classified as SC or A-6 (1) soil. The plasticity of the soil decreases as the depth increases as illustrated in the sample taken immediately below (from 2 to 4 feet). This sample is still in the B-horizon and is designated as B_2. The sample contained 31.3% gravel, 50.1%

sand and 18.6% of fines with a LL of 27.6% and a PI of 9.3%. The soil shows a CBR value of 15.0. The soil falls into the SC or A-2-4 soil classification. The C-horizon taken below the 4 foot level is a loose, coarse sandy-gravel soil (contains 49.1% gravel, 36.9% coarse sand, 5.2% fine sand and 8.8% fines). The LL of the soil is 14.7% but is non-plastic in character. The CBR test shows a value of 11.7. This parent soil is classified as SP-SM or A-1-b soil.

A sample taken on site No. 8 about one mile southwest of the previous site shows a more sandy texture. The B_1-horizon taken between a depth of 12 to 24 inches reveals the following data: 12.6% gravel, 67.1% sand, 20.3% fines, LL of 12.9%, non-plastic and a CBR of 7.4. It is classified as SM or A-2-4 soil. The soil taken between 2 to 3 feet (B_2-horizon) shows an increase of sand (76.7%) and decrease of gravel (6.3%) and fines (17.0). The soil is non-plastic and classified the same as the layer above.

The other pitted outwash plains are located east of Warsaw stretching in a southeasterly direction toward Pierceton, and south of Webster Lake on or near the eastern border. The topography is much more pitted than the plain mentioned previously. Muck kettles are numerous. Most of them are aligned with the direction of the longitudinal axis of the pitted outwash plains.

Sample No. 25 was taken about two miles northwest of Pierceton. The test data of the B-horizon taken at a depth of 2 to 3 feet from the surface shows 17.1% of gravel, 65.9% sand and 17.0% fines with a LL of 21.9% and PI of 9.0% and the soil is classified as SC or A-2-4 soil. The clean gravelly parent material taken 5 feet below

the surface contains 61.1% of gravel, 22.3% of coarse sand, 5.4% of fine sand and 11.2% of fines. It has a LL of 16.6% but is non-plastic. The parent soil is classified as SP-SM or A-1-a soil. The CBR values shown for the parent material is less than that of the B-horizon soil (12.5 and 23.5 respectively).

(2) Sandy-Gravel Outwash Plain

The largest sandy-gravelly outwash plain lies on the northern half of the county with Leesburg as its geographical center. The plain extends north through Milford into Elkhart County, terminates at Webster Lake to the east, and follows the Tippecanoe River to the west into Marshall County. The small outwash plain northeast of Lake Wawasee may be considered as a part of the same outwash plain but divided by swamps and lakes. This sandy-gravel outwash plain has a flat topography sloping very gently from the northeast to the southwest. Muck channels or pockets are comparatively few on this vast outwash plain except near to the major drainage channels and the lake chains to the east. The surface is considerably flat with depressions about 10 feet in magnitude. The pitted appearance on the airphoto is less pronounced than those on the pitted outwash plains previously discussed. Surface drainage is absent from this region except occasionally current scars are developed into faint drainage gullies. The pervious nature of the deposit is evident from the drainage map shown on Figure 3.

The pedological soil of the sandy gravelly outwash deposit is chiefly Fox soil. However, areas in the vicinity of Leesburg are mapped as Warsaw soil and Maumee organics, and Plainfield sandy soils are found adjoining the lower, organic-sand outwash plains.

Several soil samples were taken for this sandy-gravel outwash plain. The soil profile of the Fox series on the east was taken at site No. 18 located about one mile and a half south of Webster Lake. The surface soil about 6 inches in depth is generally a medium to coarse sandy soil containing very little organic material. The B-horizon taken between 1 to 3 feet from the ground surface contains 26.9% of gravel, 42.2% coarse sand, 11.0% fine sand and 19.9% fines. The minus 40 material shows considerable plasticity (LL = 35.8%, PI = 17.6%) and has a low CBR value of 2.7. It is classified as SC or A-2-6 soil. The C-horizon at the level of 3 to 4 feet shows a marked increase of gravel (57.5%) and decrease of fines (10.4%). The soil still has plasticity (LL of 23.6%, PI of 5.3%). The CBR value increases to 26.7. This parent material is classified as SW-SM or A-1-b soil.

On the western part of the plain, a sample taken from site No. 7 located about two and a half miles southwest of Atwood shows a more sandy B-horizon. It has 22.6% gravel, 43.7% coarse sand, 19.0% fine sand and 14.7% fines. The C-horizon taken between 6 and 8 feet below the surface contains only 18.8% of gravel. The amount of coarse sand increases to 65.0% and both fine sand and fines decrease to 10.4% and 5.8% respectively. Soil at both horizons reveal non-plastic characteristics. SM or A-2-4 soil and SP-SM or A1-1b soil are classified for the subsoil and parent material respectively.

The soil on the level plain near Leesburg is more silty in texture at the surface. A sample taken at site No. 16 about three and one half miles west of Leesburg exhibited a B-horizon (2 to 4 feet) with 19.5% gravel, 40.6% sand, 29.9% fines, a LL of 23.4% and

PI of 9.1%. The subsoil is classified as SC or A-2-4 soil. The gravel fraction increases in the C-horizon to 55.6%. This non-plastic soil contains only 4.4% fines. It is designated as a well graded sandy gravel SW of A-1-a soil.

The number of gravel pits on the sandy-gravel outwash plain is much less than that of the pitted gravelly outwash plain. This confirms the fact that the deposit is too sandy or not as gravelly as the pitted outwash plain.

(3) Sandy Outwash Plain

The sandy outwash plains are confined to the northwestern part and the northeastern corner of the county. Small areas are found adjacent to the larger muck channels. The surface configuration varies from smooth to very gently undulating. The pitted appearance completely disappears in this low altitude sandy outwash plains. Owing to the low topographic position and the sandy texture of the soil the surface drainage systems are not well developed. Many of the low areas next to the muck kettles are high in organic matter. The top soils are black in color containing high percentage of plant remains. These highly organic sandy outwash plains are delineated from the airphoto and designated with organic symbols on the map. Soil in this area is chiefly Maumee sandy soil A soil sample obtained from test site No. 9 located about six miles northeast of Etna Green exhibited under the highly organic-silty top soil a B-horizon (between 1 to 5 feet) with 12.1% gravel, 49.6% coarse sand, 18.3% fine sand and 20.0% fines. The soil fraction has a LL of 18.5% and PI of 1.1 and is classified as SM or A-2-4 soil. The C-horizon taken below 5 feet from the surface contains

the following: 3.0% gravel, 57.0% coarse sand, 35.9% fine sand and 4.1% fines. It is a non-plastic sandy soil. The soil is classified as poorly graded sand SP or A-3 soil.

Another test site was selected near the muck channel at site No. 14 located about three miles east of the previous site. The B-horizon taken between 1 to 3 feet shows 24.3% gravel and 25.6% of fines. The coarse sand decreases a little (33.0%) and the fine sand is approximately the same (17.1%). The plasticity (LL = 24.2%, PI = 6.9%) of the soil is higher than that of site No. 9. However, the soil falls into the same classification by the BPR system and is classified as SM-SC soil by the USCS. The C-horizon taken at a depth 3 to 5 feet from the ground surface has 7.8% gravel, 71.5% coarse sand, 17.8% fine sand and 2.9% fines. This non-plastic sandy soil fits in the same classification as that of the previous site. No gravel pit is found on these sandy outwash plains. This gives further confirmation about the sandy nature of the deposit. The subsoil and parent material after proper compaction show fair to good supporting power (CBR value varies from 3.8 to 23.8).

Outwash Plain Transition

A small area located between the rugged ridge moraine to the west and an outwash plain to the east and about three miles northwest of Pierceton is considered as an outwash plain transition. This area has a gently undulating topography. Very little surface drainage is developed in the area. The eastern part has a similar airphoto pattern with the outwash plain to the east and the western part resembles the morainic pattern to the west.

Test site No. 17 located at about the center of the area shows the granular nature of the deposit. Lenses of clean sand and gravel are found up to 12 feet deep. The B-horizon taken between 2 to 6 feet from the surface is composed of 11.0% gravel, 19.4% coarse sand, 24.0% fine sand and 45.6% fines. The LL and PI are 22.4% and 7.5% respectively. It is classified as sandy clay SC or A-4 (2) soil. The C-horizon taken 12 to 15 feet below the ground surface shows a considerable increase of gravel (29.5%), coarse sand (34.1%), about the same amount of fine sand (25.8%) and a decrease of fines (10.6%). Though the soil has a LL of 15.8% it is non-plastic. The parent material at this site is classified as SW-SM or A-2-4 soil. The bearing capacity of the C-horizon show a good CBR value of 26.8.

Terraces

The valleys of the Tippecanoe River and the Eel River mark the sites of former glacial sluiceways, which presumably carried tremendous volumes of meltwater from the Pleistocene ice sheets. As the wasting ice freed its load of soil and rock, much of this detritus was transported within tributary drainage systems and deposited within the sluiceways.

Along the Tippecanoe River huge terraces occur on both sides. Since this terrace is a continuation of the enormous outwash plain mentioned previously no further discussion should be made. Only along the small tributaries where narrow confined valley train deposits occur are considered to be terraces. Terraces occur in various levels between the stream and the upland along Eel River at the southeastern corner of the county.

The terrace deposits are similar in texture, drainage, and relief to outwash plain deposits. Gully development, if present, is incipient

and only the large streams descending from the uplands have sufficient cutting power to develop channels across the terraces. A useful criterion in recognition of the granular terrace deposits on airphotos is the disappearance at the periphery of gullies which descend from the upland.

The principal soils are those of the Fox series, associated with small areas of Plainfield and Maumee soils. Generally, clean and stratified sand and gravel are found at depths ranging from 3 to 4 feet. Such deposits are excellent sources of borrow and subgrade material.

At test site No. 4 located about three miles east from the southwestern corner of the county, the variation of the fluvial deposit is illustrated. On the highway cut a sample taken 3 feet below the surface contains 51.7% gravel, 37.5% coarse sand, 6.0% fine sand and 4.8% fines. The material is non-plastic and is classified as SP of A-1-b soil. At the near-by high ground across the road, a deep waterlaid sand deposit is found. Sands of the top 2 to 3 feet are more uniform and free of pebbles. A sample taken at 11 feet from the ground surface shows only 1.2% of gravel. However, both coarse sand and fine sand increase to 49.4% and 45.7% respectively. The soil has a LL of 13.6% and is non-plastic. It is evaluated as poorly graded sand SP or A-3 soil. This should be considered as a local variation for the terrace soil.

Recent Alluvium

Practically all drainage channels possess recent alluvial plains; however, only those of considerable size are shown on the soils map because of scale limitation.

The largest alluvial plain in Kosciusko County lies along the Tippecanoe River and its tributaries. Due to the low gradient of the stream many swamps have formed. The swamps along the Tippecanoe River

are mostly low flats not more than a few feet above the water surface. Mucky surface soils are found near the river banks. The subsoil is generally sandy.

The alluvial plains along Eel River and its tributaries are in a better drainage position. Some of the alluvial plains are 10 to 20 feet above the present channel so that recent overflow is not frequent or of long duration. Genesee soils are common for the alluvial plains. They are sandy to silty in texture. Along the major stream sandy soils are predominate. Silty soils occur on the tributaries.

WIND DEPOSITED MATERIAL

The wind (eolian) deposited material in Kosciusko County is limited to wind blown and sand only. Owing to the enormous outwash deposits in this county it is likely that the prevailing wind will pick up some sand particles and redeposit them in the nearby area. The wind blown sand deposits are generally found on the outwash plains and along the flood plain of the Tippecanoe River. Some sand dunes are noted on the airphoto especially on the pitted outwash plain north of Atwood. Some are transverse type, others are crescent like dunes. The majority of the wind blown sand deposits probably are the sheet type or incipient dune type.

The extent of these wind blown deposits are very small. Except in the dune areas they are not indicated on the map.

Plainfield is the major soil series of the wind deposit and Coloma sand is the other where wind blown sands are found overlying the drift material. The wind blown soil is generally classified as poorly graded sand SP or A-3 soil.

MISCELLANEOUS FORMATIONS

Muck and Peat

The greater part of all the organic soils of Kosciusko County may be considered as muck. The well decomposed organic matter is made possible by the lowering of the ground water table by extensive ditching in the county. Peats are found underlying the decomposed muck or in swampy land where oxidation is very slow or ineffective.

The largest muck channel is located near the northern border in the northwestern corner of the county. It is drained by the Middle Fork Yellow River to the west. The thickest muck and peat bed is about 20 feet in magnitude (11). Another large bed is located northeast of Etna Green. It was once the site of an old glacial lake basin. The depth of the bed is variable. An average thickness of 13 feet is found in the Sphagnum moss portion on the northeastern part of the deposit (11).

Muck pockets of medium size are numerous throughout the county. Small ones are concentrated along the northern slope of the Packerton moraine. Many muck or peat beds are long and narrow. The arrangement and the direction of their axis indicate, to a certain extent, the direction of flow of the glacial melt water.

The dark photo tone of the muck depressions and its lower topographic position are easily delineated from aerial photographs. Since the depth of these muck or peat deposits varies greatly, each deposit should be field investigated for design purposes.

Highly Organic Top Soils

Depressed areas where internal drainage is somewhat retarded by the high ground water table give rise to the accumulation of considerable amount of organic topsoil. There are a large number of such areas. Most

of them are located within the sandy outwash plain on the northern half of the county. They lie generally next to the muck beds or adjacent to the lakes. The parent material underlying the organic topsoil are essentially the same as those of the surrounding area. The soil in general is designated as Maumee soil. Where large areas exist, an organic symbol is used on the attached soils map to delineate the possible problem soil areas.

Swamps

There are considerable areas mapped as swamp in Kosciusko County. The largest one is located along the alluvial plain of Tippecanoe River. Due to the low lying topography mucky soils are formed overlying the sandy subsoil. Timber growth covers many shallow muck areas especially along the lower course of the river in this county. On the sandy spots, elsewhere, coarse grasses and reeds are the principal vegetation. Other swamps are scattered around the lakes on the northeastern quarter of the county. They were submerged by water and were part of the lake during the early development. As the ground water table lowered they become swamp. Many of them are actually muck beds. Appropriate precaution, therefore, should be taken in planning engineering structures across these areas.

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All airphotos used in connection with the preparation of this report automatically carry the credit line: "Photographed for Commodity Stabilisation Service, Performance and Aerial Photography Division, USDA."

Site No.	Horizon	Depth in feet	%	Unified Classification	BPR Classification
1					
	C	8		CL	A-4 (3)
2	B	3		SC	A-4 (1)
3	B	3		CL	A-6 (4)
4a					
	C	11		SP	A-3
4b	C	3		SP	A-1-b
5	B	1-2		CL	A-6 (9)
	C	4		CL	A-4 (5)
6	B			SC	A-6 (2)
	C	3-4		SC	A-4 (3)
7	B	2-5		SM	A-2-4
	C	6-8		SP-SM	A-1-b
8	B_1	1-2		SM	A-2-4
	B_2	2-3		SM	A-2-4
9	B	1-5		SM	A-2-4
	C			SP	A-3
10	B	2-3		CL	A-4 (5)
	C	3		CL	A-4 (4)
11	B	2-3		CL	A-6 (6)
	C	3-4		CL	A-6 (8)

TABLE II

SOIL TEST DATA OF KOSCIUSKO COUNTY, INDIANA

Site No.	Horizon	Depth in feet	Grain Size Distribution: Gravel % retained on #10	Coarse sand % retained #10 - #60	Fine sand % retained #60 - #200	Silt and Clay % less than #200	Liquid Limit	Plastic Index	OMC for Standard Proctor Test	Max. Dry Weight for Standard Proctor Test	CBR	Moisture Content for CBR Test	Unified Classification	BPR Classification
1														
	C	8	7.7	19.6	20.9	51.8	21.2	8.7	10.7	124.6	6.5^3	10.4	CL	A-4 (3)
2	B	3	3.2	28.7	29.8	38.3	20.6	7.7	13.7	116.2	3.5^4	13.2	SC	A-4 (1)
3	B	3	8.2	16.1	21.4	54.3	28.9	11.9	14.6	116.3	5.5^3	12.8	CL	A-6 (4)
4_a														
	C	11	1.2	49.4	45.7	3.7	13.6	NP	10.3	124.0	6.9^2	10.5	SP	A-3
4_b	C	3	51.7	37.5	6.0	4.8	NP	NP	11.0	124.9	17.6^2	11.0	SP	A-1-b
5	B	1-2	6.0	9.9	17.4	66.7	34.6	17.1	17.1	109.1	8.3^3	18.6	CL	A-6 (9)
	C	4	9.2	14.4	18.6	57.8	22.8	8.6	12.8	117.9	8.0^3	11.5	CL	A-4 (5)
6	B		19.1	18.8	18.8	43.3	27.3	10.4	14.3	113.8	10.9^3	14.2	SC	A-6 (2)
	C	3-4	14.4	25.2	20.9	39.5	22.8	9.9	10.2	125.8	5.4^3	10.0	SC	A-4 (3)
7	B	2-5	22.6	43.7	19.0	14.7	18.8	NP	12.1	120.6	5.3^3	12.0	SM	A-2-4
	C	6-8	18.8	65.0	10.4	5.8	13.6	NP	11.9	117.1	15.0^2	12.7	SP-SM	A-1-b
8	B_1	1-2	12.6	38.4	28.7	20.3	12.9	NP	10.5	109.3	7.4^2	11.2	SM	A-2-4
	B_2	2-3	6.3	39.3	37.4	17.0	16.0	NP	11.3	116.2	24.6^2	11.3	SM	A-2-4
9	B	1-5	12.1	49.6	18.3	20.0	18.5	1.1	11.7	122.3	3.8^3	13.0	SM	A-2-4
	C		3.0	57.0	35.9	4.1	NP	NP	14.2	109.0	23.8^2	15.4	SP	A-3
10	B	2-3	3.3	17.5	20.0	59.2	21.8	9.3	11.8	119.1	6.4^4	11.4	CL	A-4 (5)
	C	3	7.2	17.0	20.4	55.4	19.5	7.7	10.1	123.0	16.8^3	9.5	CL	A-4 (4)
11	B	2-3	5.4	15.4	20.1	59.1	25.5	11.9	14.4	114.0	5.3^3	14.5	CL	A-6 (6)
	C	3-4	3.9	13.9	16.7	65.5	29.6	15.0	14.5	117.1	6.7^3	13.3	CL	A-6 (8)

Site No.	Horizon	Depth in feet	Unified Classification	BPR Classification
12				
	C	4	CL	A-6 (11)
13	B	2-3	SC	A-4 (1)
	C	4-5	SM	A-4 (3)
14	B	1-3	SM-SC	A-2-4
	C	3-5	SP	A-3
15	B_1	1-2	SC	A-6 (1)
	B_2	2-4	SC	A-2-4
	C	4-7	SP-SM	A-1-b
16	B	2-4	SC	A-2-4
	C		SW	A-1-a
17	B	2-6	SC	A-4 (2)
	C	12-15	SW-SM	A-2-4
18	B	1-3	SC	A-2-6
	C	3-4	SW-SM	A-1-b
19	B	1-3	SM-SC	A-4 (3)
	C	3-5	ML-CL	A-4 (3)
20				
	C	6	CL	A-6 (12)
21	B	1-3	SM	A-2-4
	C	3-4	SM	A-2-4

Table II Continued

Site No.	Horizon	Depth in feet	Grain Size Distribution Gravel % retained on #10	Coarse sand % retained #10 - #60	Fine sand % retained #60 - #200	Silt and Clay % less than #200	Liquid Limit	Plastic Index	OMC for Standard Proctor Test	Max. Dry Weight for Standard Proctor Test	CBR	Moisture Content for CBR Test	Unified Classification	BPR Classification
12														
	C	4	2.4	7.0	11.8	78.8	29.0	14.2	16.3	113.3	3.7^4	16.6	CL	A-6 (11)
13	B	2-3	3.8	23.9	34.7	37.6	22.2	10.3	11.0	123.0	6.3^3	12.0	SC	A-4 (1)
	C	4-5	4.4	23.3	22.9	49.4	16.4	3.7	9.5	128.8	7.0^4	10.4	SM	A-4 (3)
14	B	1-3	24.3	33.0	17.1	25.6	24.2	6.9	12.0	122.1	8.6^3	12.8	SM-SC	A-2-4
	C	3-5	7.8	71.5	17.8	2.9	NP	NP	13.7	108.0	11.3^2	17.7	SP	A-3
15	B_1	1-2	19.4	23.4	18.4	38.8	30.0	14.0	13.4	114.0	12.5^3	13.5	SC	A-6 (1)
	B_2	2-4	31.3	42.7	7.4	18.6	27.6	9.3	11.4	121.9	15.0^2	11.7	SC	A-2-4
	C	4-7	49.1	36.9	5.2	8.8	14.7	NP	11.3	127.3	11.7^2	12.0	SP-SM	A-1-b
16	B	2-4	19.5	29.5	21.1	29.9	23.4	9.1	11.6	119.7	1.8^2	12.5	SC	A-2-4
	C		55.8	36.8	3.0	4.4	NP	NP	11.9	122.0	20.3^2	11.2	SW	A-1-a
17	B	2-6	11.0	19.4	24.0	45.6	22.4	7.5	12.9	117.5	3.0^4	13.2	SC	A-4 (2)
	C	12-15	29.5	34.1	25.8	10.6	15.8	NP	11.1	113.7	26.8^2	11.3	SW-SM	A-2-4
18	B	1-3	26.9	42.2	11.0	19.9	35.8	17.6	14.5	114.8	2.7^3	15.5	SC	A-2-6
	C	3-4	57.5	25.5	6.6	10.4	23.6	5.3	9.2	128.0	26.7^2	9.8	SW-SM	A-1-b
19	B	1-3	4.7	19.5	26.4	49.4	20.5	6.7	11.1	122.5	12.0^3	10.5	SM-SC	A-4 (3)
	C	3-5	17.5	13.1	18.1	51.3	19.5	6.5	10.8	123.4	13.0^4	9.8	ML-CL	A-4 (3)
20														
	C	6	1.7	3.9	6.0	88.4	36.0	13.7	18.7	107.3	3.1^5	18.7	CL	A-6 (12)
21	B	1-3	15.4	37.7	25.6	21.3	15.9	NP	10.7	123.0	38.0^2	9.3	SM	A-2-4
	C	3-4	21.8	23.7	23.9	30.6	NP	NP	8.4	130.3	22.5^3	8.0	SM	A-2-4

Site No.	Horizon	Depth in feet	% Gr re or	Unified Classification	BPR Classification
22	B	1-3		CL	A-7-6 (14)
	C	3		CL	A-6 (7)
23	B	1-2		SM	A-2-4
	C	2-3		CL	A-4 (7)
24	B	1-3		CL	A-4 (3)
	C	3-4		ML-CL	A-4 (1)
25	B	2		SC	A-2-4
	C	5		SP-SM	A-1-a

Table II Continued

Site No.	Horizon	Depth in feet	Grain Size Distribution: Gravel % retained on #10	Coarse sand % retained #10 - #60	Fine sand % retained #60 - #200	Silt and Clay % less than #200	Liquid Limit	Plastic Index	OMC for Standard Proctor Test	Max. Dry Weight for Standard Proctor Test	CBR	Moisture Content for CBR Test	Unified Classification	BPR Classification
22	B	1-3	2.7	7.4	11.8	78.1	40.2	21.6	14.0	114.0	1.2[3]	15.6	CL	A-7-6 (14)
	C	3	2.8	12.7	16.3	68.2	24.6	11.0	13.0	120.5	8.6[3]	13.8	CL	A-6 (7)
23	B	1-2	5.0	39.4	25.3	30.3	13.6	NP	10.4	127.2	15.0[4]	10.4	SM	A-2-4
	C	2-3	5.7	9.9	14.5	69.9	21.3	7.7	12.3	119.8	3.7[4]	13.6	CL	A-4 (7)
24	B	1-3	3.9	18.4	25.3	52.4	25.8	8.8	14.8	116.2	5.8[4]	14.7	CL	A-4 (3)
	C	3-4	6.0	15.8	22.9	55.3	22.8	6.3	12.4	118.4	10.8[4]	11.9	ML-CL	A-4 (1)
25	B	2	17.1	31.2	34.7	17.0	21.9	9.0	11.2	121.3	23.5[2]	9.9	SC	A-2-4
	C	5	61.1	22.3	5.4	11.2	16.6	NP	10.0	128.0	12.5[2]	9.5	SP-SM	A-1-a

CBR Notation

1 with surcharge of 10 lb.

2 with surcharge of 15 lbs.

3 with surcharge of 20 lbs.

4 with surcharge of 25 lbs.

5 with surcharge of 30 lbs.

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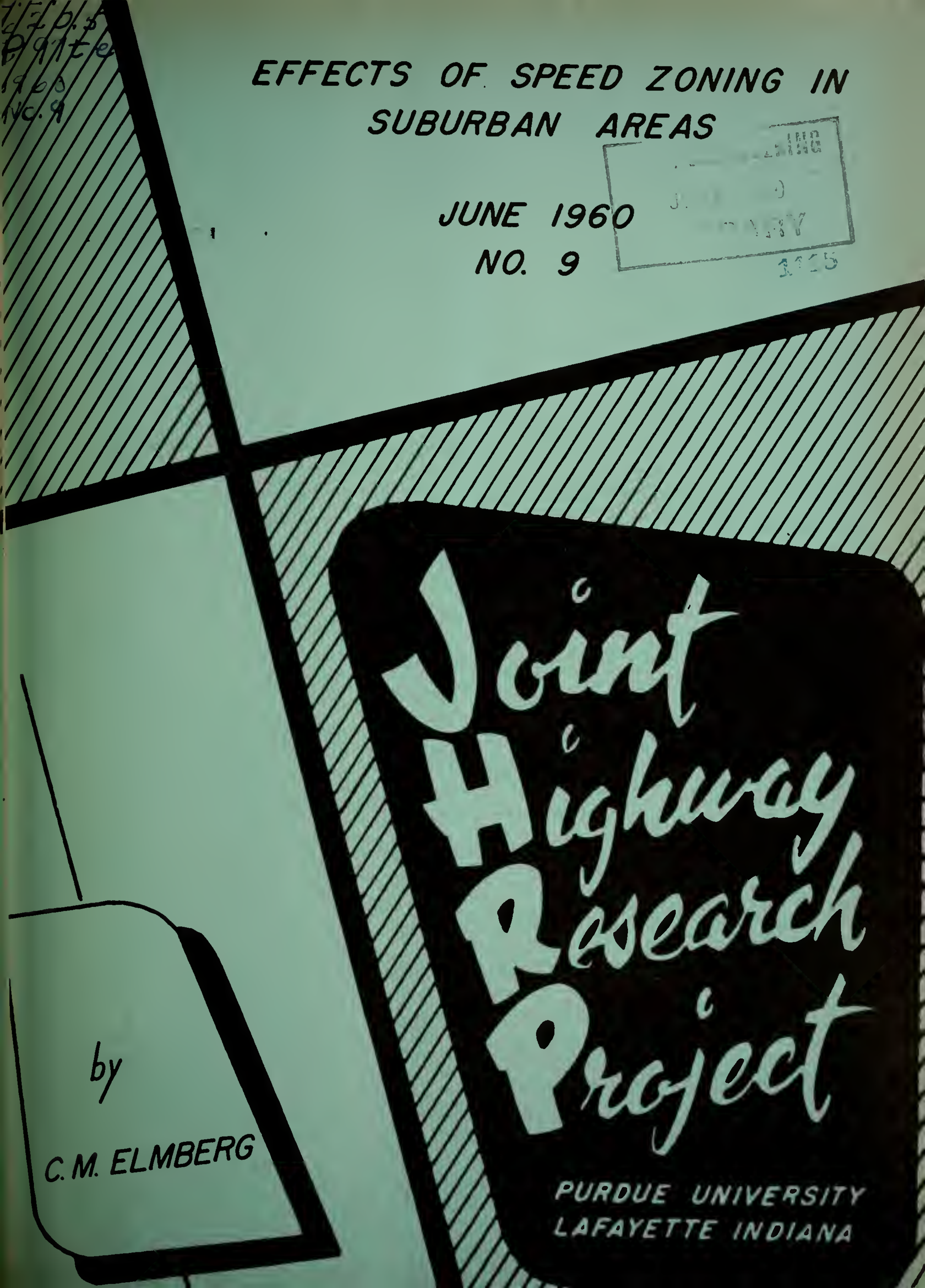
EFFECTS OF SPEED ZONING IN SUBURBAN AREAS

JUNE 1960
NO. 9



by

C. M. ELMBERG

Joint Highway Research Project

PURDUE UNIVERSITY
LAFAYETTE INDIANA

Final Report

EFFECT OF SPEED ZONING IN SUBURBAN AREAS

TO: K. B. Woods, Director
Joint Highway Research Project

June 8, 1960

FROM: H. L. Michael, Assistant Director
Joint Highway Research Project

File: 8-4-19

Attached is a final report entitled, "Effect of Speed Zoning in Suburban Areas", by Curt M. Elmberg. The research was conducted under the supervision of Professor H. L. Michael of our staff. Mr. Elmberg also used the research for his thesis for the MSCE degree from Purdue University.

The research reports that land development along a roadway does have an effect on the speed of travel but the effect was so variable that a quantitative evaluation of such effect was not possible for the various types and degrees of land development. The study also indicated that drivers on the average do not determine their speed of travel from speed limit signs but rather from their appraisal of the location and a safe speed.

The report is presented to the Board for the record.

Respectfully submitted,

Harold L Michael

Harold L. Michael, Secretary

HLM:kmc

Attachment

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EFFECT OF SPEED ZONING IN SUBURBAN AREAS

by

Curt M. Elmberg
Graduate Assistant

Joint Highway Research Project
File: 8-4-19
Project: C-36-17S

Purdue University
Lafayette, Indiana

June 8, 1960

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Acknowledgment is also made to property owners along Northwestern Avenue in West Lafayette for their interest, cooperation, and patience during the entire study; and for permission to use their driveways and lawns.

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ABSTRACT

Elmberg, Curt Martin. MSCE, Purdue University, June 1960. EFFECTS OF SPEED ZONING IN SUBURBAN AREAS. Major Professor: Harold L. Michael.

Speed control has been identified as one of the most important tools in attempting to counteract excessively fast driving and to reduce accidents. At the same time, speed control also has been recognized as a difficult and controversial issue due to the fact that criteria for establishing speed zones do not have the same degree of acceptability as have other traffic controls, for example no-passing zones or traffic signals. Numerous speed studies have been conducted throughout the United States; very few, however, deal with speed patterns within the fringe area between the genuine rural and genuine urban area, generally termed the suburban area.

This thesis reports the results of a study at five suburban areas of the combined effect of various speed limits and roadside development on driver speed patterns. Three of the locations concerned were two-lane U.S. highways traversing suburban development of predominantly residential, commercial, and industrial type, respectively. One of the locations was a four-lane, partially divided, highway which recently had been reconstructed from two lanes to four lanes. No speed limits had been posted along this highway after the reconstruction

had been completed. Consequently, this highway was an excellent location for studying the effects of various posted speed limits, making possible comparisons between unrestricted and restricted speeds. Only free-moving passenger vehicles traveling on dry pavement under optimum day and night visibility conditions were considered. Speed patterns for local and non-local vehicles were analyzed separately; local vehicles being those officially registered in the actual county as verified by the prefix on the license plate. Spot speeds were recorded, using an Electro-matic radar speed meter, at various points at each location, during day and night, before and after speed limit signs had been posted. Altogether, 31,573 vehicles were recorded, of which 20,552 were during the day and 11,021 were during the night.

The study revealed that the drivers paid little, if any, attention to posted speed limits. They seemed to choose a speed which they themselves considered appropriate for prevailing conditions. Non-local vehicles traveled significantly faster than local vehicles in many instances. Noticeable differences between day speeds and night speeds were also recorded at some sites. Variabilities in the speed distribution at a few individual stations for the same sign and road conditions were unexpectedly high, thus somewhat challenging the accuracy of fixed-point speed measurements.

The influence on the speed pattern of various types of roadside development was difficult to distinguish intelligibly, and was evidenced by the revelation of distinct interaction effects between the diverse factors. The limited number of locations, adequate for the roadside development study, also contributed to making difficult an ultimate conclusion.

INTRODUCTION

Speed

For years philosophers have argued that speed, per se, is not destructive. The elliptical path traveled by the earth with a speed exceeding 66,000 miles per hour in its annual trip around the sun is actually a highly advanced freeway. There are, however, no conflicts or frictions along this freeway in terms of crossroads and other vehicles.

To a mathematician, physicist, or astronomer speed may simply mean the ratio of distance to time; to a traffic engineer speed, per se, means nothing, but in conjunction with terms such as mean speed, speed percentiles, speed zones, and others it has a significant connotation.

Man always has strived and always will strive for attaining faster and faster speeds on the land, on the sea, and in the air. The problem today is not primarily to develop vehicles which are able to travel with high speeds but merely to develop additional devices for man which will assist him in controlling and utilizing the speeds attained safely and efficiently. Automotive vehicles have improved tremendously during the last half century. The record for the fastest mile in 1898 was 39 miles per hour and in 1950 394 miles per hour (55)*. The average driver, however, has shown very little ability to cope

---

* Numbers in parentheses refer to listings in the bibliography.

safely with high speeds. There may be safe highways and safe vehicles but no safe drivers, hence regulations and restrictions. McMonagle (44) voices this in the following quotation: "Speed is one of the great essential benefits which make highway transportation indispensable in modern America. It must be provided for and protected."

Speed and Safety

What speed is safe? Drastically expressed, the only safe speed is zero mile per hour (29, 66). Accidents occur at all speeds. However, higher speeds increase the chances of exposure to situations, the rapidity at which these develop reduces the ability of the driver to react properly and may lead to an accident. It is generally true that the speed determines the severity of an accident (17).

With the early growth of the use of motor vehicles special attention was given the problem of safe speed. In 1925 Dickinson and Marvin (4) defined safe speed as "such that a driver will be able to stop his vehicle within the distance ahead that is certain to be free from any obstruction". This is certainly true today, too. What is then a good driver? Is he the one who always drives at a safe speed? Johnston (34) defines him as "one who starting on a journey regardless of the roadway, weather, vehicle, or traffic arrives at his destination without having any trouble himself or without causing anyone else any trouble". A Yale University study of driver behavior (34) disclosed that 81 per cent of the drivers do not get involved in serious traffic accidents at all; that 15 per cent have only one accident each; and that the remaining 4 per cent account for 36 per cent of all accidents.

Another study (41) revealed that the individual speeds of the drivers with accident records are slightly higher than those for the drivers without accident records.

Yet, speed is often advocated being the greatest contributing cause to accidents. Thorough studies, however, have indicated that this may not be entirely true. A study in Minnesota of 40,000 accidents (29) in which data as to vehicle speeds were clearly stated showed that if every accident in which speed was the only violation could have been prevented the number of accidents would have been reduced less than 10 per cent. Also, if all accidents could have been prevented in which speed was the primary cause of the accident the savings in number of accidents would not have been much greater. Actually, nearly 75 per cent of the accidents involved some other violation but no speed violation. In 1955 the Chicago Park District ascribed speed as the principal cause of only 9 per cent of the automobile accidents (42). As stated previously, speed does determine the severity of an accident but is not necessarily the causative factor. "Speed is only dangerous relative to attendant conditions" (44).

Speed Control

Speed control is probably one of the most important, difficult, and controversial problems of traffic operations. Criteria for establishing speed zones do not have the degree of acceptability as have such other traffic controls as no passing zones or traffic signals. Speed control is important because it facilitates movement at uniform speed which is desirable from the standpoint of capacity and safety;

speed control is difficult because variations in driving behavior between individuals do complicate the establishment of adequate, reasonable, and, as far as possible, uniform warrants based on objective speed surveys; speed control is controversial because of divergences of opinion among engineers, enforcement officers, the motorists, and the people living along the highway as to appropriate methods for controlling speeds.

Speed control is not an invention of recent date; sections of the Roman Law cover the proper operation of chariots. Newport, Rhode Island, set up the first known speed control legislation in the United States in 1678, when a law was passed forbidding reckless and fast driving of horsedrawn vehicles. In 1757 the City of Boston limited riding on horses and driving of horsedrawn vehicles to walking speed (50). With the entrance of the automobile speed legislation became crucial. A gentleman, named T. H. Shevlin, was fined ten dollars for speeding in excess of 10 miles per hour in Minneapolis in 1902 (36). Though the speed laws passed generally decreed "speed should be reasonable and prudent for the conditions prevalent" they left the determination of such speeds to enforcing officers. In the late 1920's three kinds of speed limits had crystallized: the 'no speed limit', i.e., speed should be reasonable and prudent, the 'prima facie speed limit', and the 'absolute speed limit'. During the 1930's arguments concerning advantages and disadvantages of the respective speed limits filled magazines and newspapers throughout the nation (5, 8, 9, 10). Accordingly, speed laws were changed in various states; one system was abolished, another adopted. The general trend was towards absolute

speed limits, particularly within the cities, because such limits were generally easier to enforce (3, 55).

There is no doubt that speed control is desirable, because it assists the motorist in selecting speeds that are safe and that permit him to obtain the maximum utility, economy, and convenience from his vehicle and the road (47). Speed zoning is defined as the establishment of safe and reasonable speed limits, based upon traffic and engineering study, for certain special zones or sections of street or highway where the general state-wide legislative speed limits do not fit the road and traffic conditions (47). To be enforced speed limits must be reasonable; establishing low limits and allowing a high tolerance must be characterized as an unsound policy. Furthermore, speed limits should be established for favorable weather and traffic conditions; further reduction due to inclement conditions should still be regarded as the responsibility of the driver.

Approaching an urban area, speed zones generally begin in the fringe area, usually termed the suburban, in which auxiliary roadside development of all kinds may be expected, such as filling stations, drive-in-theaters, cafes, trailer courts, et cetera. This transition zone between rural and urban development should also be a transition zone for speed limits. Type of roadside development may or may not influence the speed pattern in this area. A reasonable signing of speed limits should reflect the gradual and natural reduction of the speed within the suburban area; accordingly, a series of speed limit signs, properly posted, gradually limiting the speed from the state-wide limit in the rural area to the limit generally used in the urban area may

constitute the best warrant available. An abrupt change in the speed limit from the high rural limit to the low urban limit within the suburban area, oftentimes at the border of the urban area regardless of development, is, however, commonplace. Very few drivers like to apply the brakes without any conceivable reason, and, therefore, the low speed limit often is entirely or partly disregarded until roadside development and inclosing traffic automatically regulate the speed.

Speed and Enforcement

A speed limit has to be reasonable in order to be enforced. An unreasonably low speed limit will generally be disobeyed, and arresting everybody violating such a limit requires a police force of dimensions inconceivable. A posted speed limit must be self enforcing. A speed limit is ordinarily not considered self enforcing if more than five per cent of the drivers exceed the limit by 5 miles per hour or more (34). Most enforcement agencies also agree that in areas where speed is a problem, apprehension of even one per cent of the drivers would be difficult.

Theoretically, if a posted speed limit is determined by an objective study enforcement should take place even though the limit is exceeded by only one mile per hour. Because of deficiencies of the equipment used in checking speeds (75) and the speedometers of the vehicles, a tolerance is customarily maintained, though not publicly known, before apprehension takes place. The most commonly used tolerance is five miles per hour (59). Notwithstanding that speedometer error is the primary reason for allowing tolerances, several other

reasons, more or less important, exist according to a study by the Eno Foundation (59). Table 1 shows some of the most characteristic reasons, discovered by this study, for allowing tolerances in enforcement of speed regulations by state as well as city enforcing agencies.

TABLE 1*

REASONS WHY TOLERANCES ARE ALLOWED IN ENFORCEMENT OF SPEED REGULATIONS

Reason	Per Cent of Reasons Reported				
	State	Cities with Population			
		10,000 - 25,000	25,000 - 50,000	50,000 - 100,000	Over 100,000
Adjustment for Speedometer Errors	56	40	36	35	41
Create Public Good Will	25	36	39	27	39
Courts Require for Conviction	16	14	17	19	13
Numerical Limits Considered Too Low	3	10	8	19	7
Total	100	100	100	100	100

* Source: Reference (59).

Purpose

The purpose of this investigation was to study the effect of speed zoning in a suburban area not previously zoned. The general effect of speed limit signs on the speed pattern, as well as some specific effects related to the posting of signs and the influence of roadside development were to be analyzed. It was also hoped that information pertaining to the establishment of adequate warrants for speed zoning in suburban areas would be obtained through this study.

PREVIOUS INVESTIGATIONS

Continuous records of speed trends before World War II are very scarce. Some states made speed surveys of fairly intensive character during the depression years of the 1930's but these studies were not repeated in a way which would have permitted year-to-year comparisons. With the restrictions on gasoline and rubber followed by a nation-wide speed limit of 35 miles per hour in 1942, the states began to make continuing large scale speed studies.

The earliest major speed-trend information came from Rhode Island, where the average speed showed a steady increase from 22 miles per hour in 1925 to 34 miles per hour in 1934 (55). Another prewar study at seven locations in New York in 1935 disclosed an average speed of 43.5 miles per hour. In 1950 the same locations were re-studied, and the average speed was then 41.7 miles per hour, accompanied by a noticeable increase in the concentration of speeds around their central values (55).

During the 1950's extensive speed studies were conducted throughout the United States. Speed has been correlated with factors pertaining to the drivers, such as sex, age, occupation; to the vehicles such as type, age, mileage; to the roads, such as alignment, width and number of lanes, type and width of medians and shoulders, among many other things. The bibliography covers this material fairly well.

Studying speed reports from various states, one notes that almost

every study was concerned with the number of drivers exceeding the posted speed limits. McClintock states already in 1931 (8): "It is generally known among competent observers that speed limits have negligible effect upon the speed at which vehicles are operated. Drivers operate at that speed which appears to them to be safe and desirable under the circumstances. Studies made in states having fixed limits, prima facie limits and no limits resulted in approximately the same average speed under comparable traffic conditions." In 1931 speed tests in Maryland and in New Jersey, where a prima facie law for 40 miles per hour was in effect, and in Massachusetts, where a 20 mile per hour prima facie law was in effect revealed that the speed chosen by the majority of drivers was substantially the same, about 35 miles per hour (10). Nearly 25 years later the problem was still the same; in a ten state survey in the eastern part of the United States 99 per cent of all drivers ignored speed limits of 15 to 25 miles per hour (62). Table 2 shows results from a speed test on a four lane divided highway (U.S. 40) in Maryland (13), in which the additional effect of a 'speed zone ahead' sign was analyzed.

Conradt (25) frankly proclaims: "Motorists generally ignore posted speed limits in New Mexico. Two likely reasons for this regard are the setting of speed limits too low and the irregular, haphazard signing of speed zones. In some communities more than 90 per cent of the drivers exceeded the posted limit; enforcement impractical, not to say impossible." Results from the tests in New Mexico (25), which included raising as well as lowering speed limits, are shown in Table 3. This study indubitably reveals that raising a speed limit close to the true 85th percentile speed does not imply that the speed will increase

TABLE 2*

EFFECTS OF SPEED ZONING IN MARYLAND

	Signs Posted		No Signs Posted
	30 MPH	Speed Zone Ahead 30 MPH	55 MPH
Average Speed (mph)	43.4	43.6	45.5
Per cent of Vehicles			
- under 34 mph (%)	5.1	5.7	3.3
- between 35 - 59 mph (%)	94.6	93.7	96.0
- over 60 mph (%)	0.3	0.6	0.7

* Source: Reference (13).

significantly after the alteration; in fact, a slight decrease is discernible in this case.

A recent study in Illinois (39), including raising and lowering of existing speed limits and establishing of limits where none previously existed, indicated that the physical increasing of the numbers on speed limit signs did not of itself generate higher speed; nor did the physical changing alter the normal speed pattern within a zone. Furthermore, no evidence existed to indicate that accidents increased when speed limits were raised, nor materially reduced when limits were lowered or established.

It is frequently claimed that lowering speed limits reduces accidents; however, a reduction of accidents at many such sites does not necessarily indicate a greater obedience of the posted limits.

TABLE 3*

EFFECTS OF SPEED ZONING IN NEW MEXICO

Community	Year	Speed Limit mph	85th Percentile Speed mph
Bernalillo	1951	20	33.5
	1953	30	31
Mesquite	1952	60	52
	1953	45	49.5
Vado	1952	60	45.5
	1953	45	45.5
Berino	1952	25	47.5
	1953	45	47

* Source: Reference (25).

Other factors such as increased enforcement activity, newspaper editorials, and safety slogans in conjunction with the alteration of a speed limit undoubtedly contribute to making the drivers more alert. A driver who knowingly violates the speed limit may also be more alert. A study in Kansas (67) verified that a reduction of accidents took place when a 35 mile per hour speed zone was installed in an area previously not zoned; yet, 70 per cent of the drivers exceeded the limit. On a four lane highway, zoned for 55 miles per hour, the number of accidents decreased after the zoning; yet, two thirds of the drivers

violated the limit. In this case it is very hazardous to conclude that the posted speed limits solely accounted for the reduction of the accidents.

A very interesting experiment regarding speed zoning was performed in 1956 in the city of Nashville, Tennessee (30). It had become apparent that the universal 30 mile per hour speed limit was too slow for some sections of the city. It was then voiced that setting the speed limits at some odd numeral such as 34 or 39 miles per hour would probably attract attention and improve the observance of the posted limits. Except for noticeable publicity when first erected by the time the speed observations were made it became evident that the effect of these signs was insignificant when compared with conventional signs indicating 35 and 40 miles per hour, respectively.

To measure deviations between posted limits and driving speeds a survey was conducted several years ago over a 500-mile section of a principal north-south U.S. route through three southeastern states (60). Although limitation were imposed, primarily by obtaining speed data using the 'floating car method' and using only a single test run, particular care was taken to attain "average conditions". Of noteworthy interest was the separation of speed data by physical conditions such as type of road and type of area; 86 per cent of the 500-mile section was rural, 7 per cent suburban, and 7 per cent urban. Table 4 shows distances posted by various speed limits, Table 5 the driving speeds related to those limits. In suburban districts the principal differences occurred where the posted speeds were 35 miles per hour or less. Where higher speed limits were indicated for suburban areas, the travel

TABLE 4*

DISTANCES POSTED BY VARIOUS SPEED LIMITS

Posted Speed Limit (m.p.h.)	Per Cent of Total Mileage (500 miles)			
	Rural Areas	Suburban Areas	Urban Areas	All Areas
15 and 20	0.1	0.6	1.0	1.7
25	10.4	3.0	2.9	16.3
30	5.2	1.0	2.4	8.6
35	3.8	1.8	0.7	6.3
40	1.0	0.6	---	1.6
45	---	---	---	---
50	53.7	---	---	53.7
55	11.8	---	---	11.8
Totals	86.0	7.0	7.0	100.0

* Source: Reference (60).

speeds were more often actually lower than the posted speed. Sixty-one per cent of the drivers violated the speed limits in the suburban areas compared to 43 per cent in the urban and 52 per cent in the rural districts.

This study distinctly reveals that the suburban area is more critical as to proper and reasonable zoning than any one of the other areas. Consequently, adequate and credible zoning of this transition area will definitely promote improved compliance even in the urban area. Although numerous speed surveys have been conducted throughout the United States,

TABLE 5[†]

DRIVING SPEEDS RELATED TO POSTED SPEED LIMITS

Posted Speed Limit (m.p.h.)	Average Driving Speed (mph)			Per Cent by Which Limits Exceeded		
	Rural Areas	Suburban Areas	Urban Areas	Rural Areas	Suburban Areas	Urban Areas
15	42	30	23	250	100	55
25	51	42	35	104	66	41
30	50	44	41	66	47	36
35	53	47	44	51	35	26
40	50	45	*	25	13	*
50	58	34	*	16	-35	*
55	60	*	*	9	*	*

* No mileage posted at these limits.

† Source: Reference (60).

very few reported concerned only the suburban area, as explicitly separated from the urban area. However, this is due to the fact that a definite borderline between the two areas is difficult, or impossible, to define in many cases. With growing urban areas the suburban area is constantly moving. A district which was defined suburban at one time might be termed urban within a few years. Alteration of established speed limits might therefore be warranted. Extending the speed limit established in the urban district to incorporate the suburban district merely from a standpoint of convenience is hardly a sound policy; nor is it sound to allow suburban development to flourish without pertinent

consideration of adequate speed control.

More research on the influence of suburbanization on traveled speeds is needed in order to ascertain warrants for establishing reasonable and enforceable speed zones. The state-wide speed limits on the open highways in the rural areas are generally reasonable. In suburban and urban areas speed limits are, in too many cases, too low and are not posted according to actual conditions, hence making a customarily law-abiding citizen a violator without any greater sense of guilt or wrongdoing.

SCOPE

Limits of Study

Some difficulty was involved in choosing the locations outside the Lafayette area at which studies were to be performed. Study sites had to be within a reasonable distance of Lafayette, carry sufficient traffic so as to expedite the collection of data and be of such a nature that comparisons of obtained data between the locations primarily reflected the influence, if any, of roadside development of various types, rather than differences due to other factors such as posted speed limits, alignment, signals, etc. Therefore, the following requirements were used for selection of a highway approaching an urban area and along which a series of speed studies were performed within the suburban region:

1. The highway had to be as straight and level as possible so as to facilitate the visual perception of the suburban as well as the urban area.
2. The posted speed limits had to be reasonable, and the limits preferably spaced in gradually decreasing order.
3. No signals, stop signs or other signs which might have an adverse effect upon the speed pattern should be present.
4. The roadside development should be predominantly of one type or another; residential, commercial, or industrial.

Considerable reconnaissance was done to find adequate locations

fulfilling the above requirements and only a few could actually be found within a reasonable distance of Lafayette.

The types of vehicles under consideration were passenger cars only because speed zoning is primarily based upon speed data obtained for these vehicles. Furthermore, since the major part of this study was concerned with changing speed limits and the study of the effects of these alterations, some thought had to be given to the residence of the vehicles. Classification was thus done into local vehicles and non-local vehicles, the former generally defined as vehicles registered in Indiana and carrying the prefix letters assigned to the county in which the study was performed. Table 6 shows the definitions used to classify the various vehicles. A separation of state-owned cars into one local and one non-local group was considered necessary because of the great number of trips by state-owned Purdue vehicles, which thus were classified as local in Tippecanoe county, where the major part of the study was carried out. Although some question could be raised about the accuracy of the classification of vehicles of type 3 in Table 6 the number of vehicles in that group is substantially smaller compared to that in other groups, hence causing little or no influence on the final result. A more intricate problem was the classification of non-local vehicles owned and operated by Purdue students. Basically, these vehicles should be classified as local in Tippecanoe county but to separate "true" non-local vehicles from such "false" ones was too difficult a task to perform. Moreover, since the study was planned to be carried out mainly during the summer months the actual number of Purdue students was reduced considerably. The reason for this concern about local and non-local vehicle classification in Tippecanoe county was the effect several

TABLE 6

CLASSIFICATION OF VEHICLES

Local Vehicles	Non-local Vehicles
1. Indiana vehicles carrying prefix letters assigned the actual county.	1. Indiana vehicles carrying prefix letters not assigned the actual county.
2. Indiana state owned vehicles which residence was known to be the actual county.	2. Indiana state owned vehicles which could not be traced as resident in the actual county.
3. Indiana vehicles carrying dealer plates.	3. Indiana vehicles carrying amateur radio plates, veteran disability plates, and similar special plates.
	4. Out of state vehicles.

alterations of sign conditions might have on drivers who were familiar to the environment. A familiar driver may notice a sudden change of a previously existing speed limit and be alert whereas an unfamiliar driver perceives the posted speed limit like any other speed limit, obeying or disobeying it.

A study of the effect of speed zoning in general cannot be regarded comprehensive unless speed zoning and visibility are related. Therefore, at all locations speed studies were also performed during night hours. It was believed that a similar classification of the vehicles during the night as was done during the day should be possible but several preliminary surveys disclosed it to be so difficult that the accuracy of the night studies might be disputed. These surveys thus

indicated that, on the average, every three vehicles had either no light illuminating the license plate or the light was in such a bad condition that it was impossible to read the license plate, particularly when the speed was high. Accordingly, no classification was done during night hours.

Only free-moving through vehicles were recorded, a free-moving vehicle having been defined for the purpose of this study as one whose speed was not affected by the immediate presence of other vehicles in its path. In accordance with this definition only the lead vehicle in a platoon was recorded. The minimum allowable headway for recording was about five seconds on a two lane highway (49) but slightly less on a four lane highway, where it oftentimes had to be judged by the recorder from case to case whether or not a passing vehicle was free-moving. Passing vehicles on a two lane highway and vehicles turning off the highway after passing the speed recording station within a distance of that station which was deemed to have affected the speed were not recorded. Moreover, since the studies were performed in the near vicinity of urban areas quite a few police cars were patroling the highways. Neither those cars nor any vehicles in the front of or in the rear of them were recorded (18). However, in a sense all vehicles were actually recorded since the volume of vehicles passing the speed measurement station was of particular interest, this volume thus consisting of vehicles whose speed was recorded and vehicles whose speed was not recorded due to the factors mentioned above.

In order to reduce the number of variables data were collected on weekdays only and, as far as possible, at the same times each day.

No night studies were carried out on Fridays because the characteristics of travel those nights was significantly different from other weekdays, particularly during the summer months (46).

Data collection took place only when the pavement was dry and optimum atmospheric conditions, absence of fog or haze, were present. A preliminary study was performed dealing with the effects upon the speed on dry and wet pavement to test whether a light rainfall during a speed survey would affect the results significantly. The study disclosed that a light rainfall did not affect the speed significantly in daylight; a light rainfall was defined as one which wetted the surface gradually but not so heavy that the windshieldwipers had to operate. Even a light rainfall at night caused reductions in speed mainly due to glare from reflected light on the wet pavement. This has also been found in investigations by other authors (57, 61). Hence, the actual study was not intended to investigate this matter in detail; merely to obtain some information in order to justify whether or not an already initiated survey should endure in case of rain.

The study was principally associated with the inbound flow of traffic towards the urban area although the outbound flow was partially studied in one suburban area. With very few exceptions, at the same speed measurement station, day and night studies took place the very same day. In some cases, efforts were also made to check the speeds continuously at some stations on the same weekday. However, great success was not accomplished because of the large number of stations covered.

Reconnaissance studies and preliminary surveys for determining

sample size were conducted in December 1958 and the first months of 1959. The actual collection of speed data took place from March through November 1959 with uninterrupted weekly surveys from May through September. Altogether, about 31,500 passenger vehicles were checked in this study.

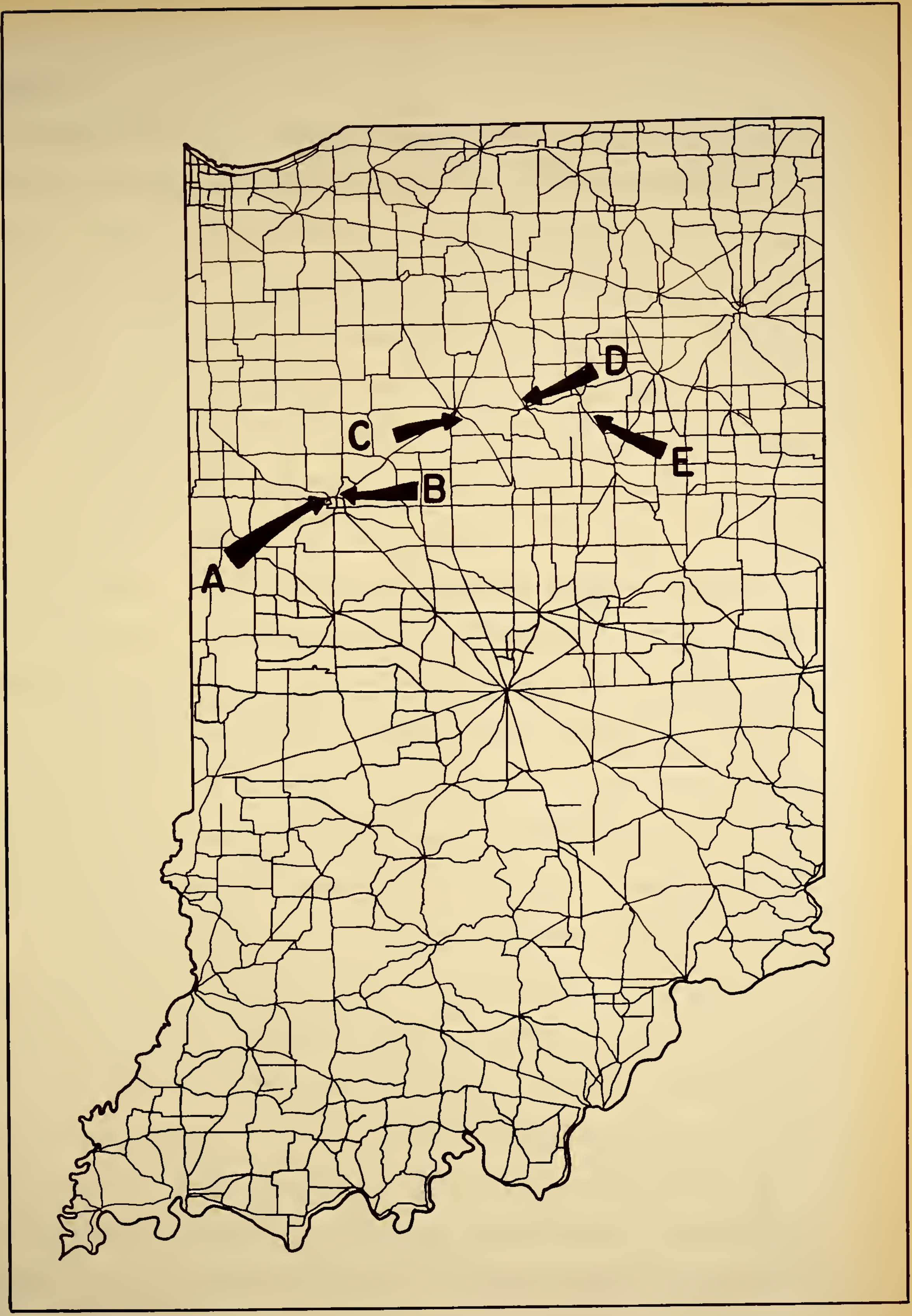
Locations

The warrants for selecting study locations, previously mentioned on page 17, limited the number of usable sites although several urban areas were visited. Only a few of them satisfied the warrants specified. The following locations were selected, the physical data of which are given in Table 7. Figure 1 shows the five locations on a road map of Indiana.

TABLE 7

SPEED STUDY LOCATIONS

Location	Urban Area	Highway	Approach to the Urban Area	Direction of Study	ADT
A	West Lafayette	U.S. 52	North	Southbound Northbound	4600-5500
B	West Lafayette	Ind. 43	North	Southbound	2800-4300
C	Logansport	U.S. 35	South	Northbound	3500-4100
D	Peru	U.S. 31	North	Southbound	4100-4300
E	Wabash	U.S. 24	East	Westbound	4300-4500



STUDY LOCATIONS

FIGURE 1

Location A.

Among all the five locations this one was chosen for the most comprehensive study due to the fact that, in the fall of 1958, the highway had been reconstructed from a two lane facility to a four lane one. No speed limits had been posted since the completion of the reconstruction. Permission was obtained from the State Highway Department of Indiana to utilize this highway for extensive signing research.

The highway, U.S. 52 Business Route, starting at the intersection of U.S. 52 and U.S. 52 Bypass north of West Lafayette, penetrates the downtown area of the cities of West Lafayette and Lafayette. From the intersection referred to above to Lindberg Road, a distance of 3,800 feet, the highway is a four lane divided facility with a 30 foot wide grass median on the first 1,000 feet, and a narrow concrete median of 5 foot width on the remaining 2,800 feet. The lane width is 12 feet. On the west side of the highway are golf courses, on the east side some development, a drive-in-theater and a smaller residential area. The development is not closely situated to the highway. Subsequently, the effect of this development upon the speed was deemed negligible. From Lindberg Road and towards the center of West Lafayette the highway has four undivided lanes, 12 foot wide, and runs through a highly developed residential area with development literally to the curb. From Northridge Drive sidewalks exist. The test section of this highway is approximately 9,000 feet long, from the U.S. 52 Bypass intersection to the Purdue University Fieldhouse. 3,800 feet runs through a primarily undeveloped area, and 5,200 feet through a highly developed residential area. The border between these two areas is very distinct, following

Lindberg Road in an east-west direction. Along 1,600 feet, between Hillcrest Road and Victory Drive, there is a 3 per cent grade, downward in the southbound direction. At Northridge Drive and at the fieldhouse there are two curves with 1,000 foot and 400 foot radii, respectively. It is quite obvious that the latter curve will affect the speed considerably. Accordingly, no studies were carried out beyond this curve. Moreover, about 300 feet beyond the curve is a signalized intersection which is not visible before entering the curve. An aerial photograph of the entire test area is shown on Figure 2.

Location B.

State Road 43 parallels the Wabash River at the north approach of West Lafayette. The lack of distinctive development along the highway in the river valley is noteworthy. Because of topography and frequent flooding only a few houses are actually situated alongside the highway. The study section comprised about 4 miles of length. The road width is 20 feet. Speed limits are assigned and include a 50, a 40, and a 30 mile per hour zone. The 50 mile per hour zone is about 3 miles long, the 40 mile per hour zone comparatively short, 300 feet, and the 30 mile per hour zone about 500 feet. A curve with 600 foot radius follows immediately beyond the 40 mile per hour sign in the inbound direction.

Location C.

The suburban development along the south approach of U.S. 35 into Logansport consists primarily of some residential development on the west side, and on the east side a stock yard, a cafeteria and a miniature golf course, a combination of industrial and commercial development.

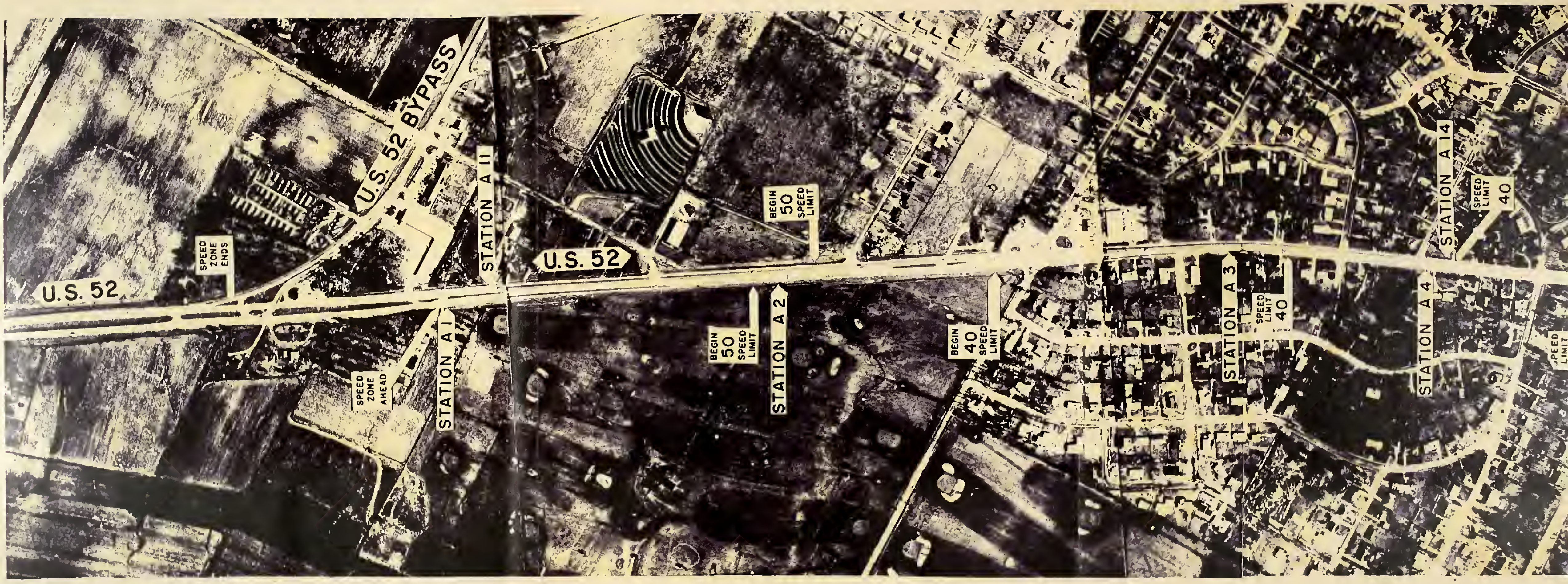


PHOTO MOSAIC OF LOCATION A

FIGURE 2

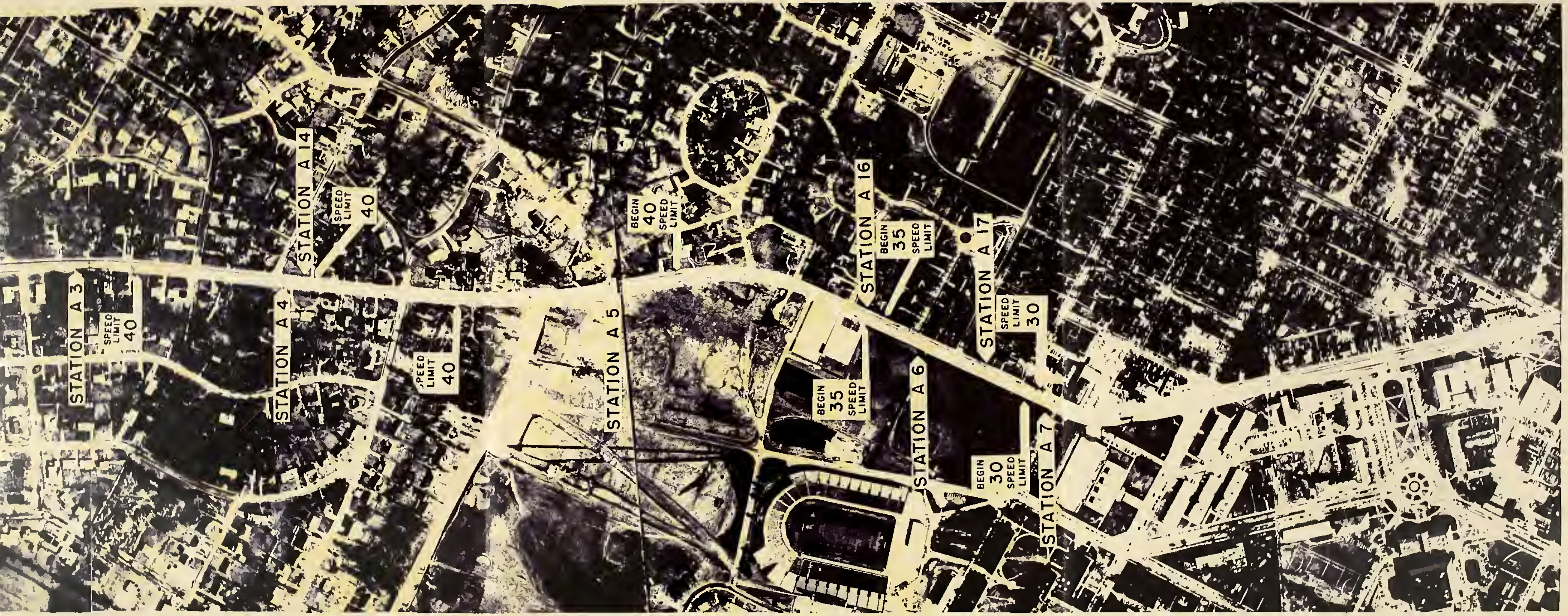


PHOTO MOSAIC OF LOCATION A

FIGURE 2

The roadway has two lanes, each one 12 foot wide. The study area comprised about one mile of length, including a 40 mile per hour and a 30 mile per hour speed zone.

Location D.

The development along the north approach of U.S. 31 into Peru is mainly commercial. The highway is perfectly straight, and the city is visible at a distance of more than a mile before reaching the city limits which coincide with the 30 mile per hour speed limit. A 40 mile per hour zone also exists for a distance of about 2,000 feet.

Location E.

Along the east approach of U.S. 24 into Wabash the development is entirely industrial, a chicken farm, a state highway garage, factories. From the 40 mile per hour speed limit sign the highway is straight and the city visible. The roadway has two, 12 foot wide, lanes. At the city limit a 30 mile per hour zone starts.

The locations A and B are specific and cannot be compared to any other location because of design features such as number of lanes, lane width, and also because of specific roadside development. Moreover, the study area for location A penetrates rather deep into the urban development. On the other hand, locations C, D, and E are similar in design characteristics except for the roadside development. Accordingly, the influence of roadside development upon the speed may or may not be significant.

Speed Measurement Stations

It is extremely important that the obtained speed data reflect the true speed of the vehicles. The drivers should not have the slightest feeling that their speed is being checked. If he has this feeling he doubtlessly will reduce speed, particularly when a speed limit is knowingly exceeded.

At every location selected a thorough detailed study was made in order to find suitable sites for measurement stations. The necessity of hiding the equipment as well as the recorders was urgent and each station had to provide an unobstructed view of the highway so as to be able to classify the passing vehicles by reading the license plates. Moreover, measurement stations should not be located too far beyond an existing speed limit sign if the effect of that sign was going to be studied. By using an artificial mailbox in which the transmitter was hidden and a panel truck, provided by Purdue University, as the recording site it was hoped to attain optimum reduction of the drivers' suspicion. It was also assumed that a relatively large number of vehicles exceeding the actual speed limit during the speed check would indicate little or no suspicion.

Table 8 briefly shows some physical data for the various speed measurement stations. Figures 3 - 8 picture the locations, generally viewed from the exact site of the stations. In cases where a speed limit sign is perceptible in the picture the site is situated immediately beyond the sign. An aerial photograph of the entire location A with all signs and measurement stations shown is Figure 2.

Because of the construction of a new bridge over the Wabash

TABLE 8

DATA RELATED TO SPEED MEASUREMENT STATIONS

Location	Measurement Station No.	Direction	Speed Limit Sign Checked	Figure No.
A	1	Inbound	SZA*	2, 3
	2	"	50	2, 3
	3	"	40	2, 3
	4	"	40	2, 3
	5	"	40	2, 3
	6	"	35	2, 3
	7	"	30	2, 4
	11	Outbound	50	2, 4
	14	"	40	2, 4
	16	"	35	2, 4
	17	"	30	2, 4
B	1	Inbound	65	5
	2	"	SZA	5
	3	"	50	5
	4	"	50	5
	5	"	40	5
	6	"	30	-
C	1	Inbound	65	6
	2	"	40	6
	3	"	30	6
D	1	Inbound	65	7
	2	"	SZA	7
	3	"	40	7
	4	"	30	7
E	1	Inbound	SZA	8
	2	"	40	8
	3	"	30	8

* SZA = Speed Zone Ahead.

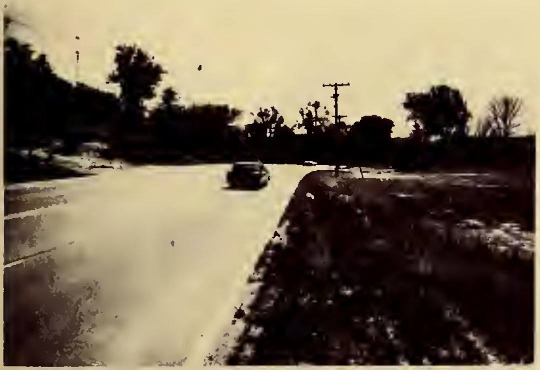
river station B6 could not be studied. Heavy construction equipment, excavation, and some other factors contributed to a general reduction of the speed of the vehicles passing that area.

STATION A 1

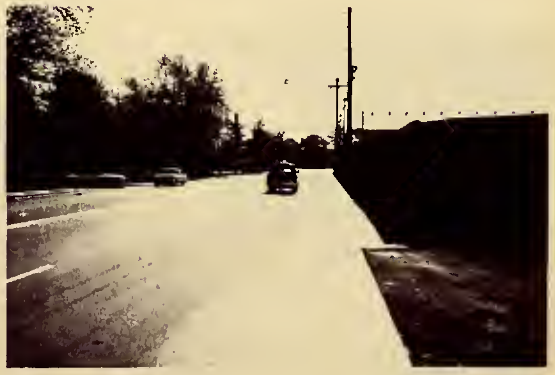
STATION A 2

STATION A 3

STATION A 4



STATION A 5



STATION A 6

LOCATION A

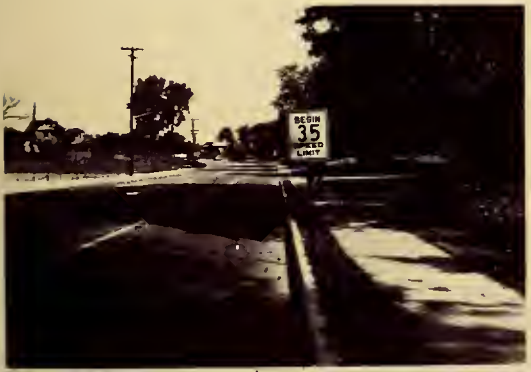
SPEED MEASUREMENT STATIONS

FIGURE 3

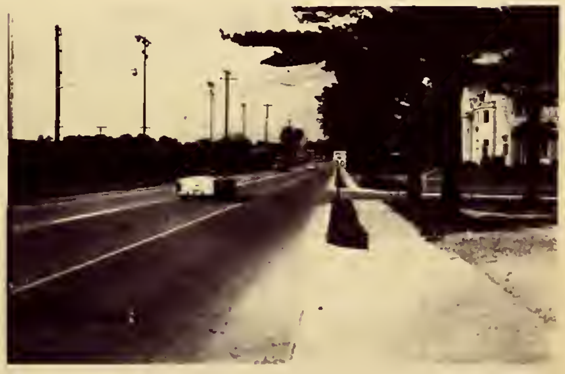
STATION A 7

STATION A 11

STATION A 14



STATION A 16



STATION A 17

LOCATION A

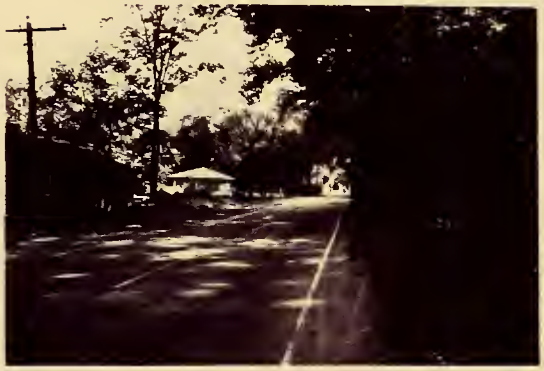
SPEED MEASUREMENT STATIONS

FIGURE 4

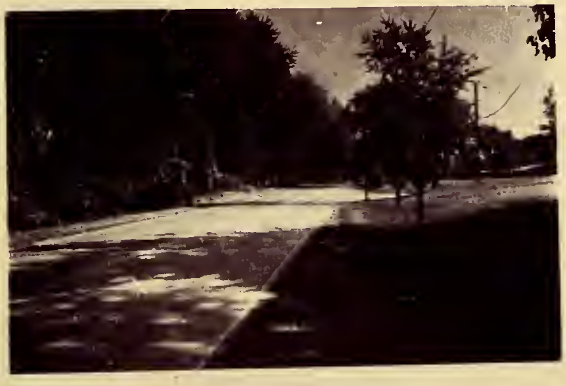
STATION B 1

STATION B 2

STATION B 3



STATION B 4



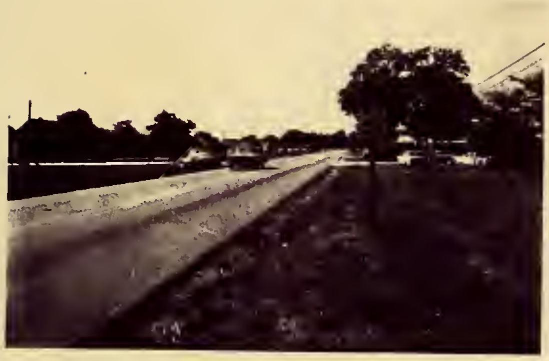
STATION B 5

LOCATION B

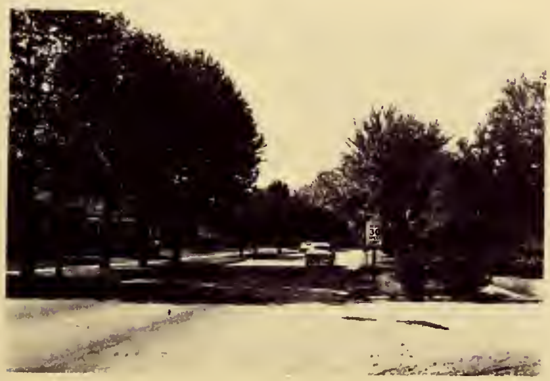
SPEED MEASUREMENT STATIONS

FIGURE 5

STATION C 1



STATION C 2



STATION C 3

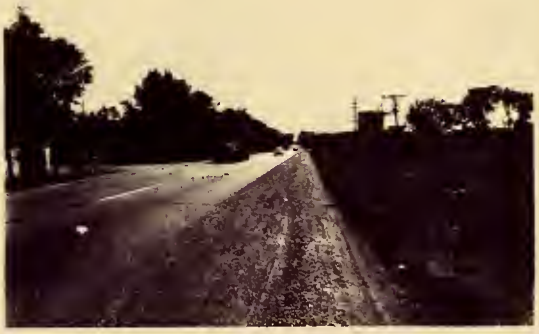
LOCATION C

SPEED MEASUREMENT STATIONS

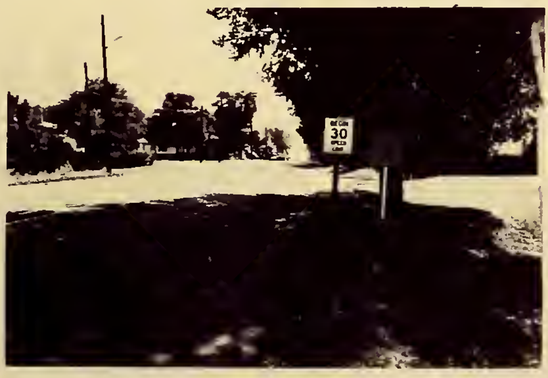
FIGURE 6

STATION D 1

STATION D 2



STATION D 3



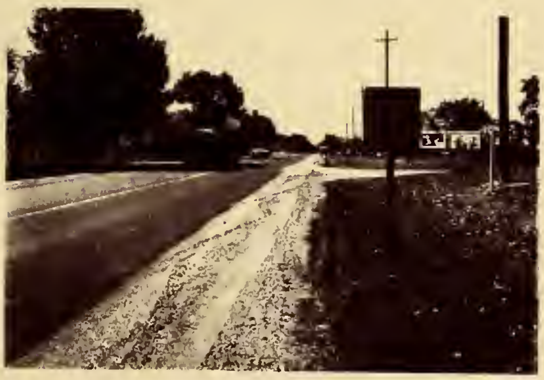
STATION D 4

LOCATION D

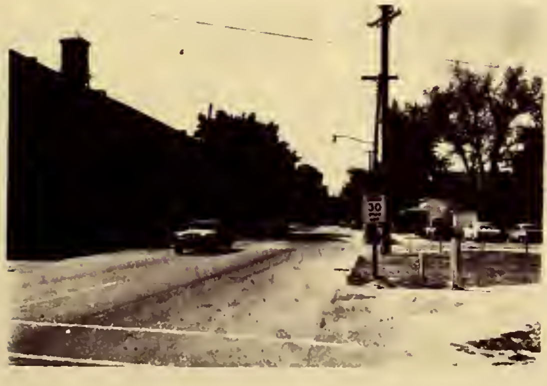
SPEED MEASUREMENT STATIONS

FIGURE 7

STATION E 1



STATION E 2



STATION E 3

LOCATION E

SPEED MEASUREMENT STATIONS

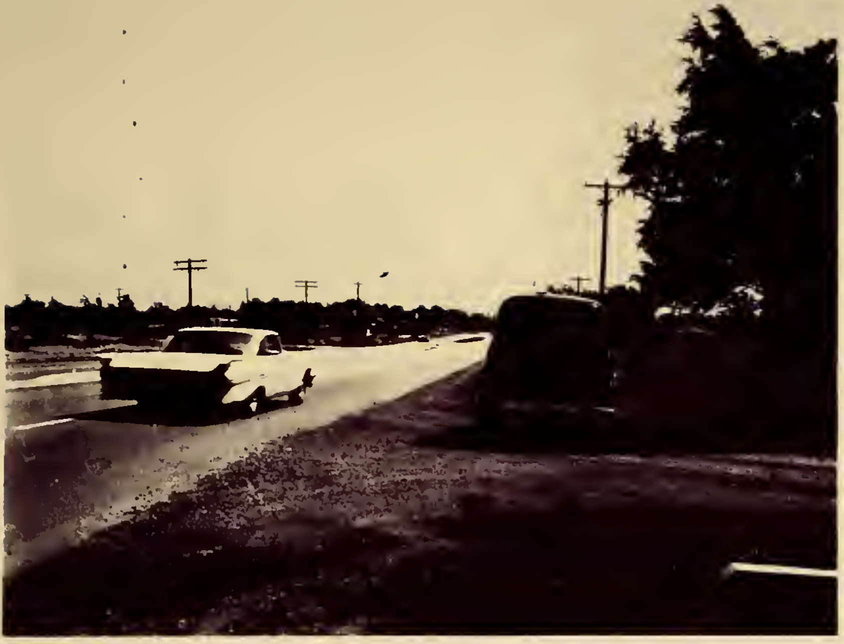
FIGURE 8

EQUIPMENT

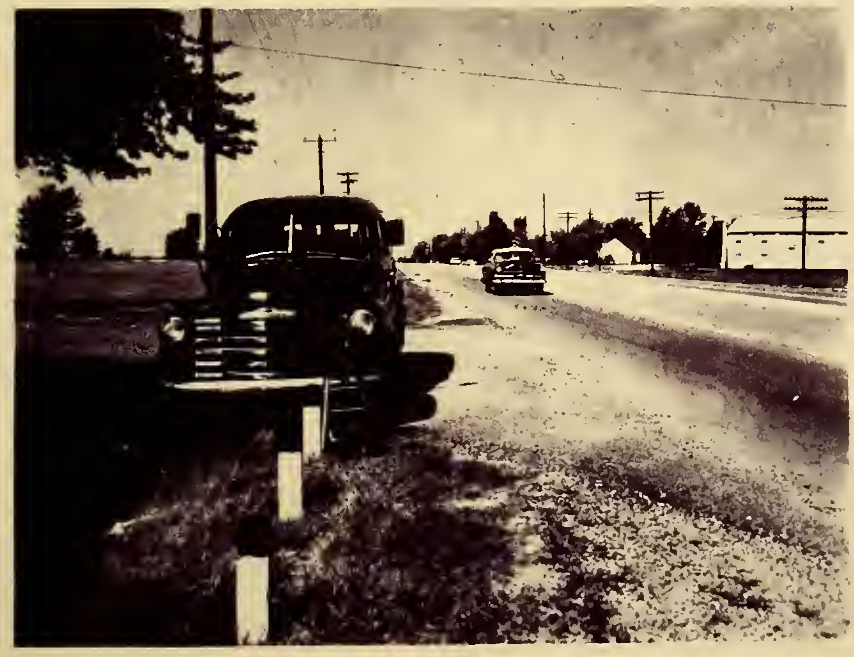
All speed data for this study were obtained by the use of an Electro-Matic speed meter powered by a 12 volt automobile storage battery. This meter belonged to the Joint Highway Research Project.

The radar meter utilizes the Doppler effect to measure speeds. Radio waves of constant frequency are broadcast by the transmitter-receiver in a cone shaped beam about 20 degrees wide. When reflected from a moving object these waves change frequency in proportion to the component of speed in the direction of the waves. The difference in frequency between the transmitted waves and the waves reflected back to the unit induces a potential that is read on the indicator, a voltmeter calibrated in equivalent miles per hour.

The effective range of the meter increases with its height above the roadway, the optimum height being about three feet. In order to achieve this height and facilitate aiming, the unit was usually supported on a wooden stool which was levelled by means of stakes laid flat beneath the legs (Figure 9). However, at some locations the unit had to be set in a higher position, approximately 5 feet above the ground, because of the use of an artificial mail box in which the unit was inserted (Figure 10). On the other hand, at a few locations the unit was set up only 1.5 feet above the ground. The main reason for this variation of position height was the ultimate necessity of



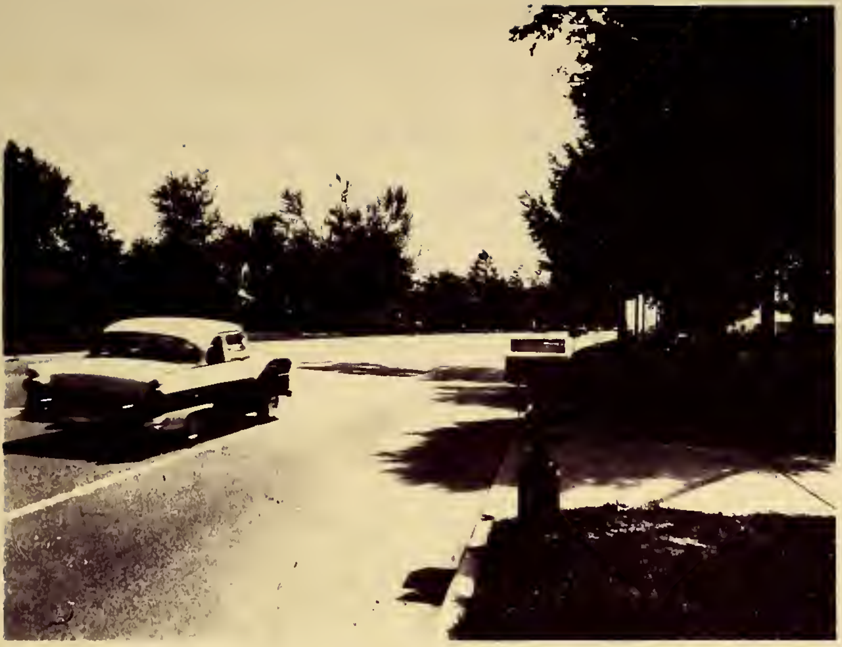
DRIVER'S VIEW



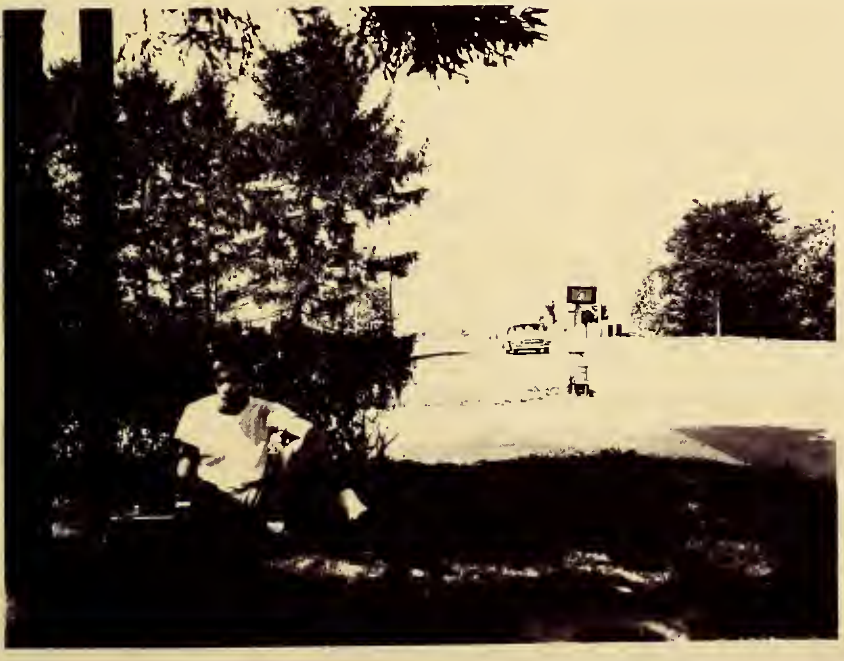
RECORDING SPEED

RADAR SPEED METER SETUP

FIGURE 9



DRIVER'S VIEW



RECORDING SPEED

RADAR SPEED METER SETUP

FIGURE 10

concealing the equipment thus avoiding suspicion and curiosity which might have caused a deliberate reduction of the traveled speed. Test runs with a Purdue vehicle equipped with a calibrated speedometer, thus providing "true" speeds, indicated no significant difference between the true speed and the meter registered speed due to the variations in height positions of the unit, at least for the actual speed range, 20 to 70 miles per hour.

The meter registers the speed of the vehicles in the beam, traveling either towards or away from the unit, to an accuracy of one to two miles per hour. However, since the range of the beam for passenger cars is only about 100 to 150 feet, depending upon the strength of the impulse, certain errors may enter into the indicated reading (75).

Meter readings were affected by the lateral position of the unit because of the necessity of setting the meter back from the pavement edge in order to minimize any effect its presence might have on traffic approaching the control station. The beam, therefore, had to be angled into the traffic stream. As a result, the indicator readings represented only a component of the true vehicle speeds. Furthermore, passenger cars in the far lane would be within range only while intersecting the beam at a comparatively large angle. Due to the fact that the main part of this study was concerned with a four lane road, Northwestern Avenue in West Lafayette, Indiana, special precautions had to be taken regarding the accuracy of the registered speed of cars in the inner lane.

A mathematical relationship of the location of the radar unit with respect to the roadway and speed and which is comprised of two simultaneous equations can be developed:

$$y = S \tan(\beta + 10^\circ) - w,$$

$$\cos \beta = \frac{V}{V + e},$$

where y = lateral distance in feet from radar unit to road edge

S = effective range in feet

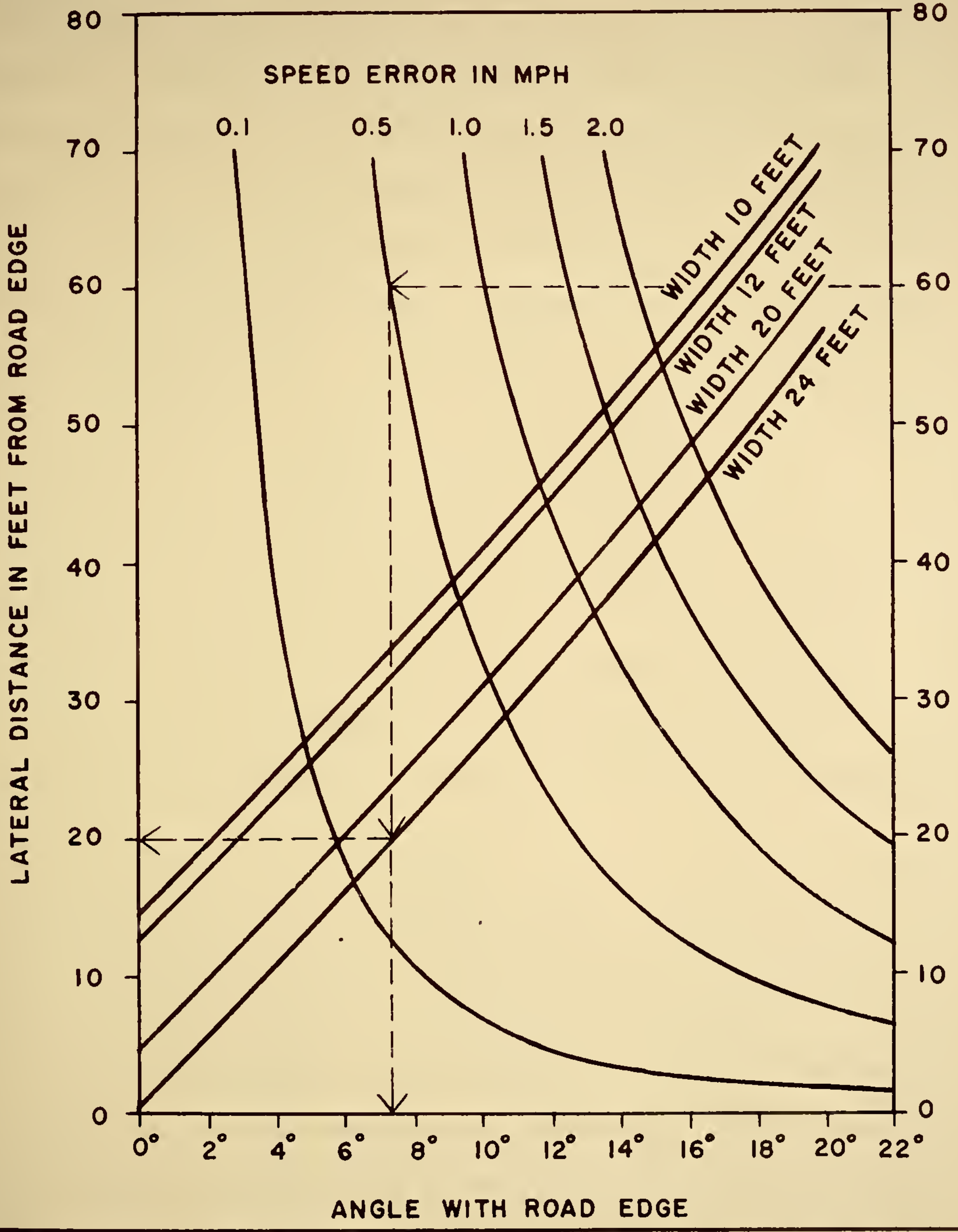
β = angle in degrees with road edge

w = width in feet of lane or roadway, respectively

V = measured speed in miles per hour

e = speed error in miles per hour

Several studies indicated that the average effective range for passenger vehicles was 140 feet. A graphical representation of this relationship is given in Figure 11. An illustrative example explains the use of this chart for selecting a proper position of the radar unit. The radar meter measures a component of the true speed, and first, the maximum allowable speed error must be decided. For example, a speed error of 0.5 mile per hour indicates that the true speed is 40.5 miles per hour when the measured speed is 40 miles per hour. It must be borne in mind that the total allowable error is comprised of the angular speed error and the statistical error, the size of which depends upon sample size and dispersion. The latter error may be of substantial size, and, therefore, the speed error should be kept as small as possible. In the usual situation, one wants to know where to place the radar unit in order to measure the speed unidirectionally for two 12 foot lanes of a four lane undivided highway, and, at the same time, avoid interference of vehicles traveling in the opposite direction. Anticipating a maximum speed of 60 miles per hour at the station studied and a speed error



POSITION OF RADAR SPEED METER

FIGURE 11

of maximum 0.5 mile per hour the unit may advantageously be placed 20 feet from the road edge and angled about 7 degrees with the road edge, in accordance with Figure 11. Any combination of distance and angle along the 24 foot line below this point is also appropriate and results in a speed error less than the presumed 0.5 mile per hour. Exactly the same result is obtained if one wishes to record speed in both directions simultaneously on a 24 foot wide road. In general, the closer the radar meter is to the edge the less is the speed error. During this entire study the radar unit was placed and concealed as close to the road edge as practicable in order to substantially limit the angular error.

Meter readings were also affected by the charge in the batteries. More than five hours continuous operation was sufficient to discharge the batteries to the point where they would no longer be reliable. Since the longest continuous operation usually lasted for only three hours whereupon the batteries were recharged no errors due to discharge were encountered.

Test runs with the Purdue vehicle equipped with a calibrated speedometer at speeds from 20 to 70 miles per hour, in increments of ten miles per hour, were performed at the beginning and the end of the study. There was no indication of any difference between the readings, neither between the two test occasions nor between the indicated and the true speed, although the meter had been used for about 500 hours between the two tests. A slight difference of 0.5 to 1 mile per hour was encountered for the 60 and 70 mile per hour test runs but the difficulty in sustaining these high speeds exactly might partially explain the difference.

At the beginning of every single speed study a tuning fork was utilized for the calibration of the meter. This tuning fork vibrated at a frequency equivalent to that which would register 60 miles per hour on the indicator.

PROCEDURE

Field

The field procedures for locations B through E were all the same. The equipment was set up on the sites selected and operated during the hours prechosen from the results of preliminary surveys. The speed was measured after the vehicle had passed the transmitter which consequently was directed to transmit the beam in the direction of the traffic flow. This method was regarded superior to letting the traffic approach the transmitted beam, thus obtaining an indicator reading before the vehicle passed. Several factors contributed to this decision, the most important ones being the readiness of hiding the transmitter behind various features and the possibility of reading the license plate and speed of the same vehicle simultaneously, thus increasing the accuracy of recording. It has to be borne in mind that Indiana vehicles only have one license plate. Two recorders were used, one reading the indicator and one reading the license plate. The latter one recorded the speed on a tally sheet, specially prepared for succeeding calculations.

The procedure at location A varied due to the fact that no speed limit signs initially existed. The speeds were first measured at all the stations without any signs posted. On the basis of these surveys the 85th percentile speed was calculated and selected as a warrant for a reasonable speed limit. The 85th percentile speed is generally

agreed to be the most reasonable in establishing a speed zone (48, 68) although one of the purposes of this study was to verify this. A signing plan was made and only one sign was added at a time. A detailed compilation of the various sign conditions are found in Appendix A. The effect of each sign was then studied by speed measurements at all stations which would be affected by it. Because of the great number of measurement stations and sign conditions there was unfortunately not sufficient time to make studies at all the sites for every change in a sign condition. A change in a sign condition consisted of posting an additional sign; with one exception, no signs were removed or changed after once added. Generally, a speed study was performed one or two days before the posting of an additional sign was scheduled. A few days after each new sign was erected the first study was made, and thereafter studies were conducted weekly in order to determine any change in speed by lapse of time.

All day studies in the inbound direction at locations A and B were performed between 1:30 p.m. and 3:00 p.m., all night studies between 8:30 p.m. and 10:15 p.m. In the outbound direction at location A, day studies were carried out between 10:00 a.m. and 11:30 a.m. No night studies were made in this direction, nor in the inbound direction on Friday nights because of variations in travel characteristics (46). Accordingly, no studies were conducted Saturdays and Sundays.

At the other locations, C, D, and E, the time of study varied somewhat due to travel time from Lafayette. However, at all locations speed measurements were taken during off-peak volume hours. The number of free-moving vehicles during peak hour traffic is small, particularly

on two lane highways. At locations B through E traffic was only checked in the inbound direction.

Several attempts were made to read the license plates during the night but had to be characterized as a failure because of inadequate or lack of illumination of the plates on an unexpectedly high percentage of vehicles. Moreover, the visibility decreased rapidly with increasing speed of the passing vehicles.

The total number of passenger vehicles, classified into local and non-local vehicles during daylight, passing each speed measurement station was manually counted and divided into two groups, one comprising the vehicles for which speed was checked and the other comprising the vehicles which, for various reasons, could not be checked. Vehicles of the latter type were either braking, turning, passing or going to perform such maneuvers and consequently could not be regarded as free moving.

As previously mentioned local vehicles were those carrying prefix letters assigned to the county in which the speed study was carried out. Table 9 lists these prefixes for the various counties.

Office

The data recorded on a field tally sheet were reworked, summarized, and transferred to a form specially designed to facilitate the first step of the statistical analysis, which comprised calculation of mean speed, median speed, 85th percentile speed, 15th percentile speed, speed differential, 10 mile per hour pace, and standard deviation. Definitions of statistical terms are given in Appendix A. The second

TABLE 9

PREFIX LETTERS ON INDIANA VEHICLE LICENSE PLATES

Location	County	Prefixes
A	Tippecanoe	MN, MN, MP, MQ
B	Tippecanoe	MM, MN, MP, MQ
C	Cass	SS, ST
D	Miami	ZX, ZY, ZZ
E	Wabash	VA, VB

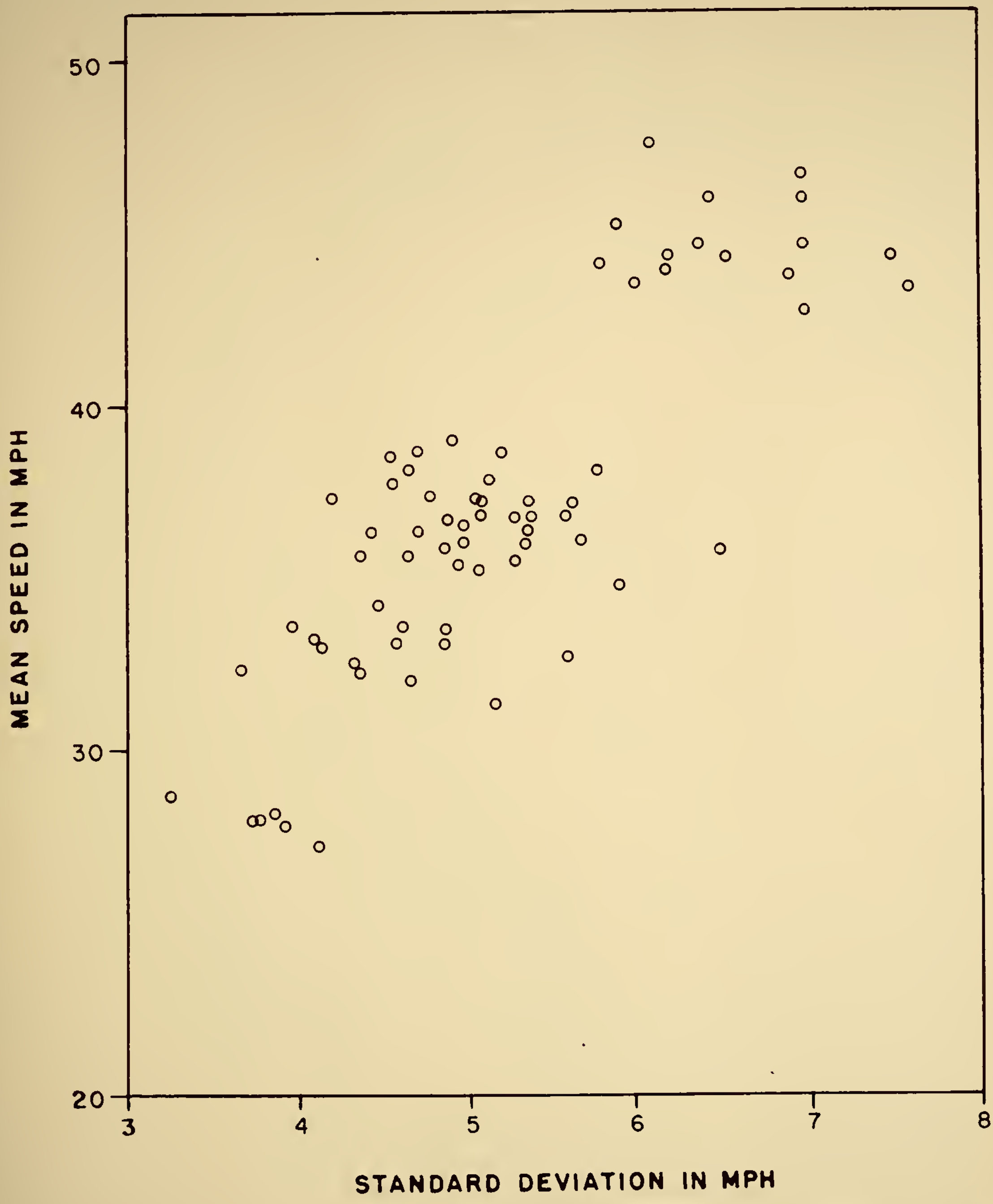
step was filing all calculated data by various influencing factors in tables (Appendix B). The third and last step was the analysis of the data in Appendix B by using diverse statistical methods and inferences, epitomized in Appendix C. An illustrative example of the calculations is also carried through in this appendix.

STATISTICAL ANALYSIS

Analysis of Speed Parameters

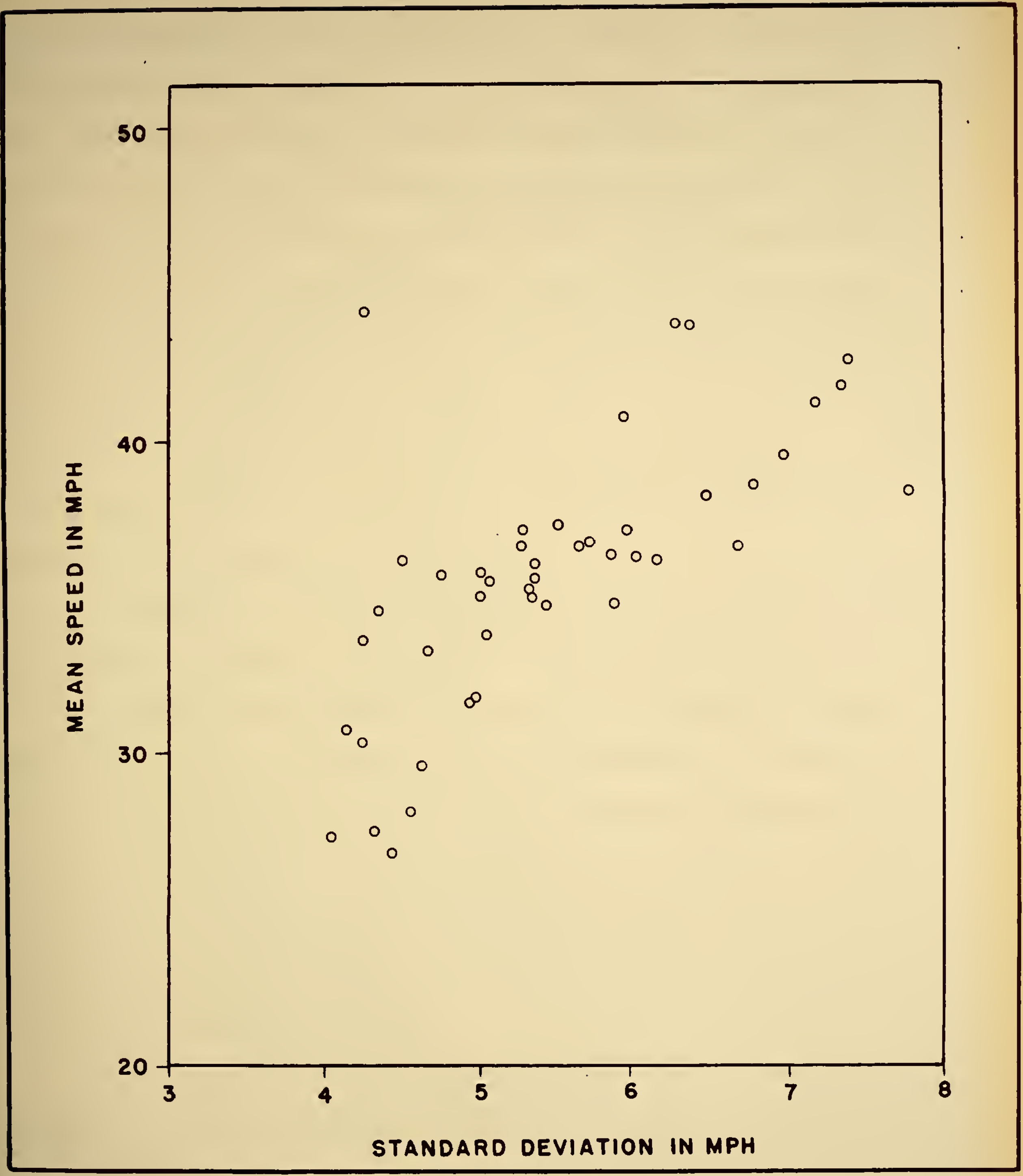
The field data were processed to yield values of speed parameters for each station by sign condition, residence of vehicle, and time of study (day or night). These parameters were a measure of two characteristics of the speed distribution: the magnitude and the dispersion. The parameters of magnitude were the mean speed, the median (or 50th percentile) speed, and the 15th and 85th percentile speed; those of dispersion were the variance (or square of the standard deviation), the speed differential and the pace.

Since the speed differential and, consequently, the standard deviation were expected to be larger in an area where a high speed was permitted than in one with a lower speed allowed, the first phase of the analysis consisted of an investigation of the homogeneity of the variances. Accordingly, standard deviations were plotted against mean speeds for day and night conditions, shown in Figures 12 and 13, respectively. It is evident that a relationship exists between mean speed and standard deviation, hence indicating heterogeneity of the variances. In analyzing the data the statistical method termed "analysis of variance" (ANOVA) was utilized. The following basic assumptions related to the data are connected with this method and govern its usefulness: additivity of effects, normality of errors, independence of



SCATTER DIAGRAM OF STANDARD DEVIATION AND MEAN SPEED AT LOCATION A (DAY)

FIGURE 12



SCATTER DIAGRAM OF STANDARD DEVIATION AND MEAN SPEED AT LOCATION A (NIGHT)

FIGURE 13

errors, and homoscedasticity (homogeneity of variances). The obvious heterogeneity would therefore indicate a limitation in utilizing the ANOVA, although the method is relatively insensitive to this factor compared to other ones. One method to diminish the influence of heteroscedasticity is transformation, which basically means a conversion of every original single speed measurement to some mathematical function of the same speed depending upon the kind of relationship existing between mean speed and standard deviation. Then the whole procedure has to be repeated; the transformed speed parameters are calculated and with these new data analysis of variance can be performed. A proper transformation would be logarithm of the speed.

The method just described is tedious and was not utilized in this case. From the figures 12 and 13 three distinctive clusters can easily be distinguished, each one related to one or more characteristic speed measurement stations. Consequently, the ANOVA method can be utilized for all factors affecting the speed at each individual measurement station. It is quite obvious from the scatter diagrams that there is a significant difference in speed between the measurement stations. The variations between the measurement stations were found to be far above the 5 per cent significance level.

Provisions could have been made for taking the same number of measurements for each sign condition at each individual station in order to simplify the analysis, i.e., the same number of replications per cell. The large number of conditions and stations and the limited amount of time, however, required variation in the number of replications; at some stations only one measurement was taken for some of the

most important sign conditions, at other stations several measurements were taken for the same sign condition. The reason for the latter case was to determine whether or not a lapse of time would cause any change in the speed pattern, and a few of the stations were chosen as control stations.

Because of the unequal number of replications per cell a special method was used to obtain only one replication and eventually make corrections for it in the analysis. The method is as follows (86):

Let X_i = calculated variable, either average speed $\bar{V}$ or 85th percentile speed, V_{85}

n_i = number of vehicles on which the value of X_i is based.

i = 1, 2, . . ., g replications per cell

k = number of cells

The first step is to calculate the weighted average, $\bar{X}$, given by

$$\bar{X} = \frac{\sum_{i=1}^{g} n_i X_i}{\sum_{i=1}^{g} n_i}$$

The second step is to calculate the sum of squares, SSg, within an individual cell, given by

$$SSg = \sum_{i=1}^{g} n_i X_i^2 - \frac{\left[\sum_{i=1}^{g} n_i X_i\right]^2}{\sum_{i=1}^{g} n_i}$$

Using an automatic or a semi-automatic calculator the following expression facilitates the calculations,

$$SSg = \sum_{i=1}^{g} n_i X_i^2 - \bar{X} \sum_{i=1}^{g} n_i X_i$$

If this sum of squares is divided by (g - 1), the degrees of freedom, an estimate is obtained of the mean square, MSg, based on individual measurements,

$$MSg = \frac{SSg}{g - 1}$$

The third step is to calculate the total sum of squares, SSk, within all cells, given by

$$SSk = \sum_{k} SSg$$

Dividing SSk with the sum of the degrees of freedom for each cell an estimate is procured of the mean square, MSk, based on individual measurements,

$$MSk = \frac{SSk}{\sum_{k} g - 1}$$

The fourth and last step in the procedure is to calculate the error mean square, MSe, which primarily is a correction of MSk because of the reduction of the replications within the cells,

$$MSe = \frac{MSk}{\bar{n}}$$

$\bar{n}$ is the harmonic mean of all the n_i's within the cells, the harmonic mean defined by

$$\bar{n} = \frac{1}{\frac{1}{k}\sum_{k}\left[\frac{1}{\sum_{i=1}^{g} n_i}\right]}$$

In the ANOVA table (see Appendix C) the error mean square is MSe and the degrees of freedom are $\sum_{k}(g - 1)$.

Using this method one has to bear in mind the general relationship between the standard deviation of the distribution of sample means and the standard deviation of the population, defined by

$$\sigma_{\bar{x}} = \frac{\sigma}{\sqrt{n}}$$

This relationship is confined solely to the distribution of sample means. For the distribution of sample percentiles the following relationship exists,

$$\sigma_p = K_p \cdot \frac{\sigma}{\sqrt{n}}$$

where

$$K_p = \frac{\sqrt{p(1-p)}}{f(\varsigma)}$$

For a given fraction p or percentile, i.e., $P = 100p$, the values of $f(\varsigma)$ and K_p are given in Table 10.

TABLE 10

CORRECTION FACTORS FOR PERCENTILES

p	$f(\varsigma)$	K_p	K_p^2
0.50	0.3989	1.253	1.5700
0.75	0.3178	1.362	1.8550
0.80	0.2800	1.429	2.0420
0.85	0.2330	1.533	2.3501
0.90	0.1754	1.710	2.0241

Accordingly, the standard deviation of the distribution of the 85th percentile speeds is defined by

$$\sigma_{85} = 1.533 \cdot \frac{\sigma}{\sqrt{n}}^{*}$$

Using the method of calculating the error mean square, previously outlined, the precise expression for the MSe is

---

* The variance of the distribution of the 85th percentile speeds is defined by

$$\sigma_{85}^2 = 2.3501 \cdot \frac{\sigma^2}{n}$$

Parameter	MSe	Degrees of Freedom
Mean Speed	$\frac{MSk}{\bar{n}}$	$\sum_k (g - 1)$
85th Percentile Speed	$\frac{1}{2.3501} \cdot \frac{MSk}{\bar{n}}$	$\sum_k (g - 1)$

This method yields reliable results as long as the variations within the cells are small. If the variations are great the SSg becomes substantial and consequently MSe increases reducing the possibilities of exhibiting significant results. Although the data, in terms of weighted means, may look conspicuously significant at a first glance, the variations within the cells may be of such a magnitude that there is even a significant difference between data within the cells. In the rare case of this type, the analysis probably will not yield significant differences between factors expected to do so thus indicating that the weighted means are not reliable.

Analysis of Speed Violations

This part of the analysis deals solely with vehicles exceeding the posted speed limits by 5 miles per hour or more, generally regarded as the enforcement level (59). Because the variables in this analysis are fractions, analysis of variance cannot be utilized directly. A procedure of transforming the data must first be introduced. The following notations are used:

m = the number of vehicles exceeding the posted speed limit by

5 miles per hour or more for a particular sign condition

n = the number of vehicles passing the posted speed limit for a particular sign condition

g = the number of measurements (replications) for a particular sign condition

The fraction z is defined by

$$z = \frac{m}{n}$$

Analysis of variance can then be carried out with a transformed variable u defined by (82)

$$u = 2 \text{ arc sin} \sqrt{z}$$

With an unequal number of replications per cell the same method can advantageously be utilized as has been previously outlined for pure speed data. Table 14 in Appendix B deals with all the transformed data and gives the values of z and u for each condition.

A tabulated summary of the results of the entire statistical analysis in terms of significant or insignificant changes for the various sources of variation is given in Table 11.

TABLE 11

SUMMARY OF STATISTICAL ANALYSIS

Source of Variation	Station A 1			Station A 2		
	$\overline{V}$	V_{85}	u	$\overline{V}$	V_{85}	u
Condition c(v)	NS	NS	NS	NS	NS	NS
Vehicle v	NS	NS	NS	NS	NS	NS
Interaction c x v	NS	NS	NS	NS	NS	NS
Condition c(t)	NS	NS	NS	NS	NS	NS
Time t	NS	NS	NS	S	S	NS
Interaction c x t	NS	NS	NS	S	NS	NS

Source of Variation	Station A 3			Station A 4		
	$\overline{V}$	V_{85}	u	$\overline{V}$	V_{85}	u
Condition c(v)	NS	NS	NS	S	NS	NS
Vehicle v	S	S	NS	S	S	NS
Interaction c x v	NS	NS	NS	NS	NS	NS
Condition c(t)	NS	NS	NS	NS	NS	NS
Time t	NS	NS	NS	NS	S	NS
Interaction c x t	NS	NS	NS	NS	NS	NS

Notations used:

$\overline{V}$	Mean speed	(v)	Vehicle analysis
V_{85}	85th percentile speed	(t)	Time analysis
u	Transformed fraction (see page 57)		
S	Significant change on a 5 per cent significance level		
NS	Insignificant change on a 5 per cent significance level		

TABLE 11 (continued)

SUMMARY OF STATISTICAL ANALYSIS

Source of Variation	Station A 5			Station A 6		
	$\bar{V}$	V_{85}	u	$\bar{V}$	V_{85}	u
Condition c(v)	S	NS	NS	NS	NS	NS
Vehicle v	S	S	NS	NS	NS	NS
Interaction c x v	NS	NS	NS	NS	NS	NS
Condition c(t)	NS	NS	NS	NS	NS	NS
Time t	NS	NS	NS	NS	S	NS
Interaction c x t	NS	NS	NS	NS	NS	NS

Source of Variation	Station A 7			Station A 11		
	$\bar{V}$	V_{85}	u	$\bar{V}$	V_{85}	u
Condition c(v)	NS	NS	NS	NS	NS	NS
Vehicle v	NS	NS	NS	-	-	-
Interaction c x v	NS	NS	NS	-	-	-
Condition c(t)	NS	NS	NS	NS	NS	NS
Time t	NS	NS	NS	-	-	-
Interaction c x t	NS	NS	NS	-	-	-

Source of variation	Station A 14			Station A 16		
	$\bar{V}$	V_{85}	u	$\bar{V}$	V_{85}	u
Condition c	NS	NS	NS	S	S	S

TABLE 11 (continued)

SUMMARY OF STATISTICAL ANALYSIS

Source of Variation	Station A 17		
	$\overline{V}$	V_{85}	u
Condition c	NS	NS	NS

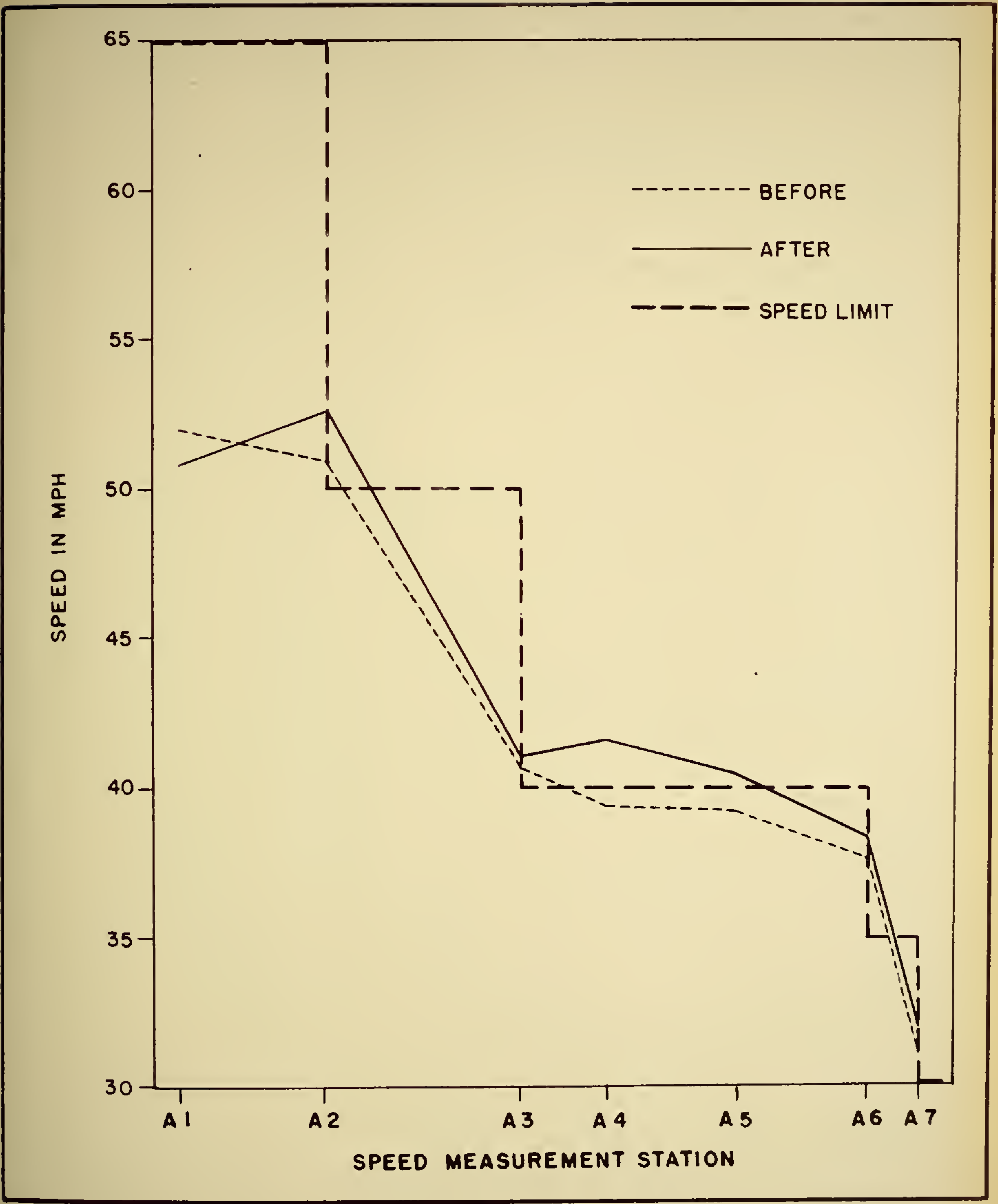
Source of Variation	Locations C, D, and E		
	$\overline{V}$	V_{85}	u
Location L(v)	S	S	S
Condition c(v)	S	S	S
Vehicle v	S	S	S
Interaction L x c	S	S	S
Interaction L x v	NS	NS	S
Interaction c x v	NS	NS	S
Location L(t)	S	S	S
Condition c(t)	S	S	S
Time t	S	S	NS
Interaction L x c	S	S	S
Interaction L x t	S	S	S
Interaction c x t	NS	S	NS

RESULTS

The results obtained by the statistical analysis are briefly outlined in Table 11, showing significant or not significant differences between all the various factors involved. A more detailed analysis of the results for each speed measurement station will be given in this chapter.

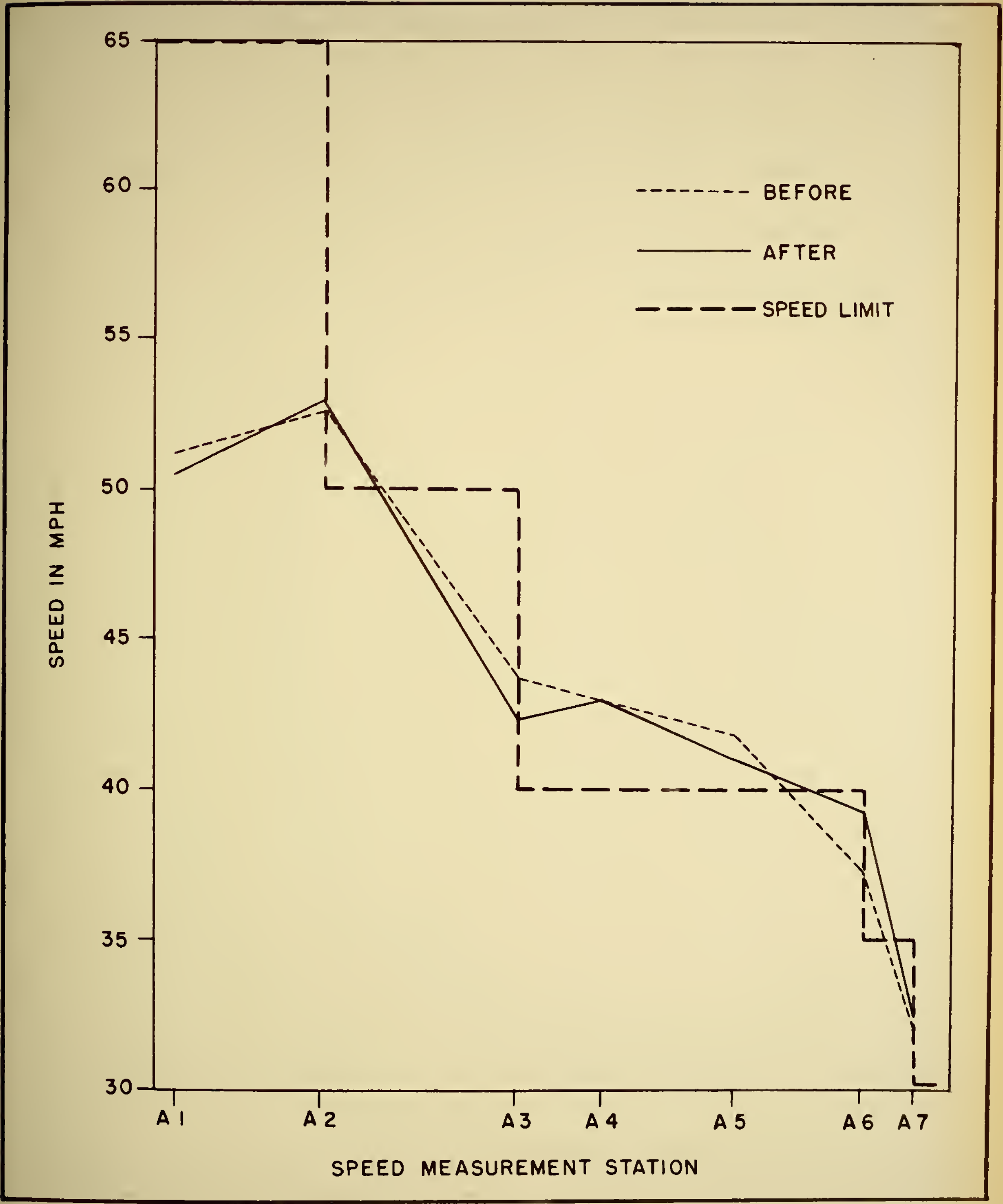
One very important point will be stated before the detailed analysis. The mere absence of speed limit signs in the developed area of location A after the reconstruction of the highway did not establish an abnormal speed pattern.

The general result of posting speed limits on the 85th percentile speed are shown in Figures 14 - 17 for local vehicles, non-local vehicles, day and night conditions, respectively. It may be noted that the speeds of local vehicles (Figure 14) are slightly higher after the posting of the signs. Part or the entire reason may be found in the removal of an uncertainty about what speed limit actually existed along this particular highway. On the other hand, non-local drivers seemed to adjust their speeds slightly closer to the signed condition (Figure 15). Comparing day and night conditions a somewhat closer adjustment seemed to be realized during night, at least for speed limits 40 miles per hour and above. The traveled 85th percentile speed is higher than the posted speed limits at all stations except for A 1, where the "Speed Zone Ahead" sign permits vehicles to travel 65 miles per hour. For



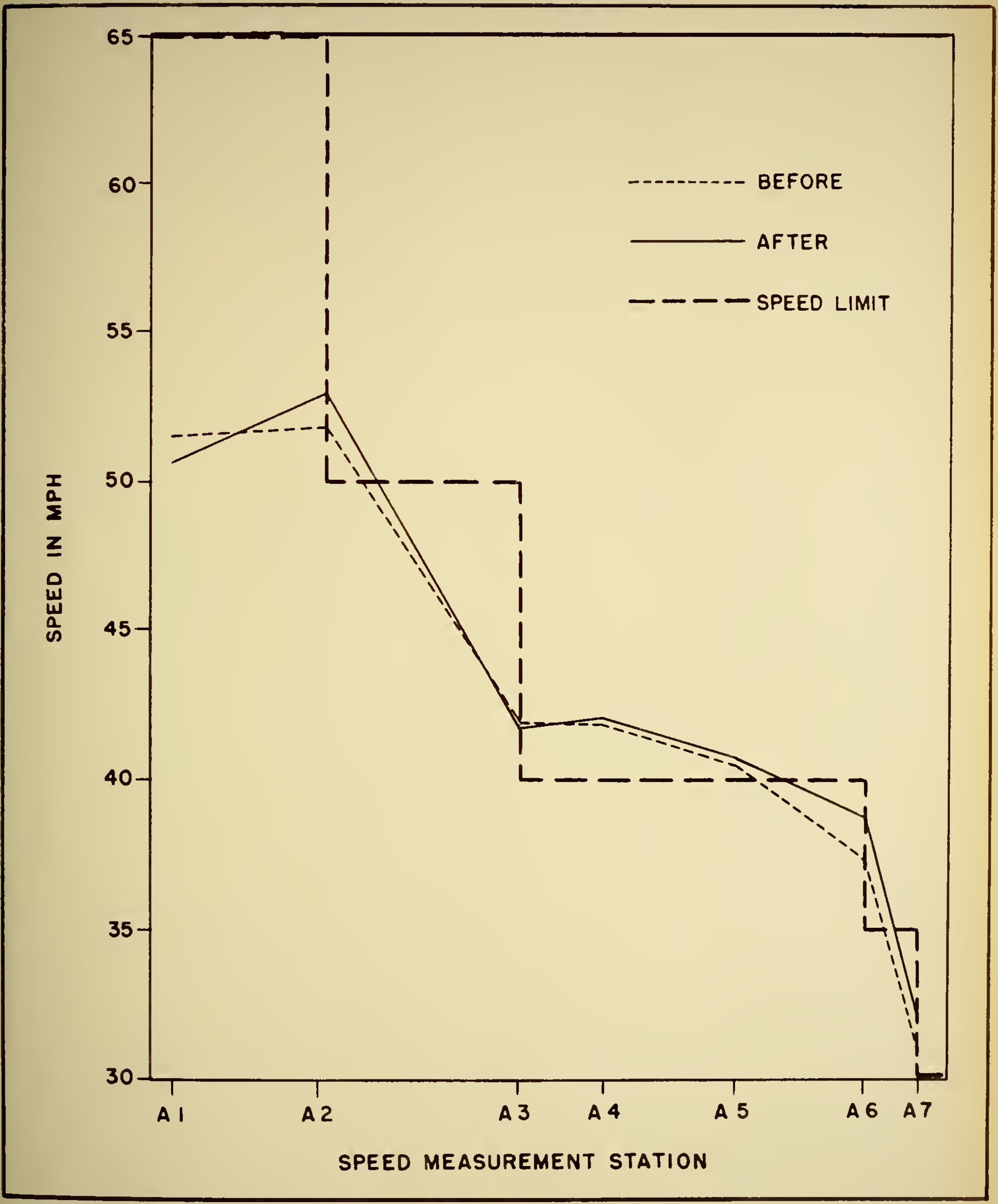
85TH PERCENTILE SPEED BEFORE AND AFTER
POSTING OF SPEED LIMITS
(LOCAL VEHICLES)

FIGURE 14



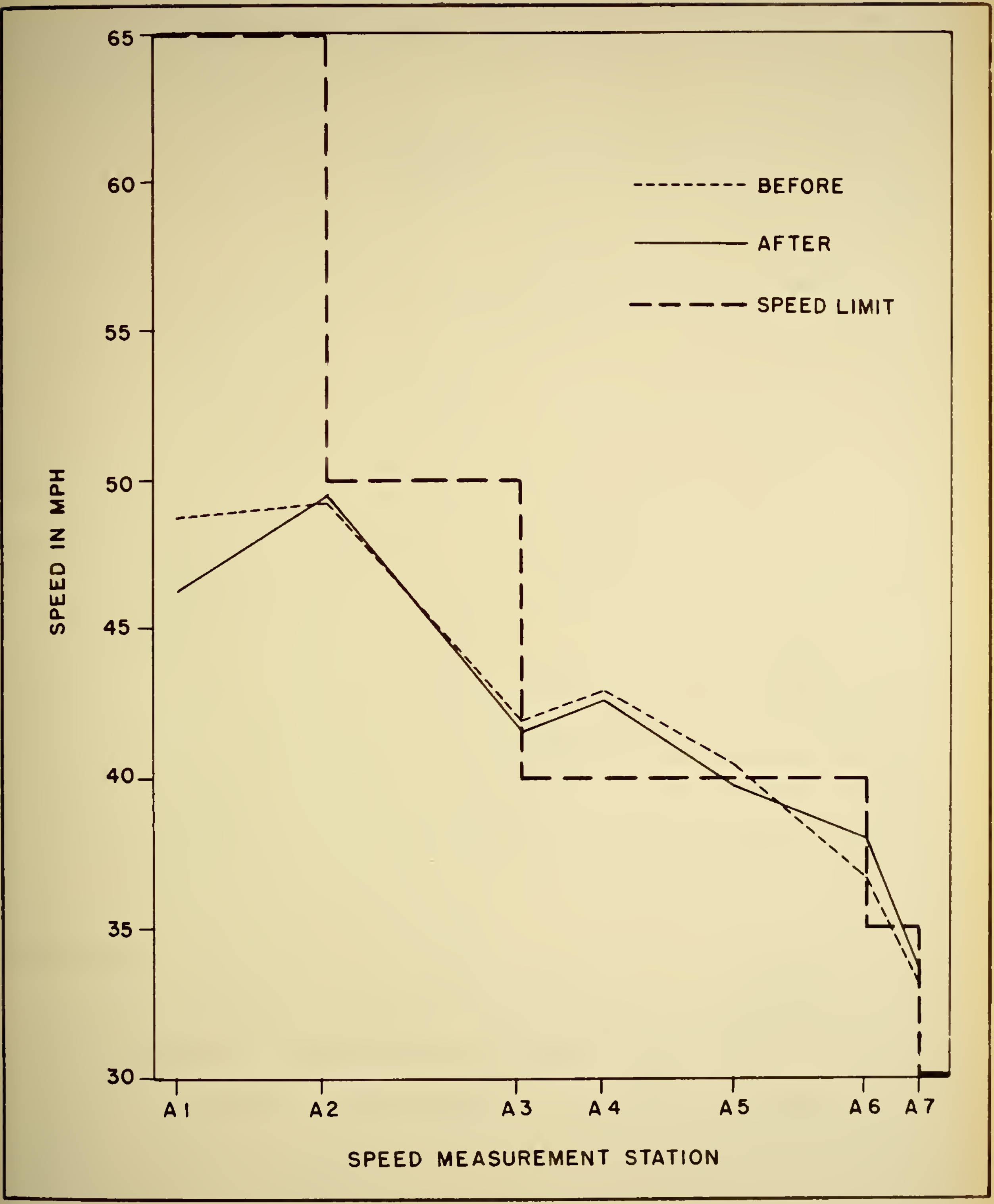
85TH PERCENTILE SPEED BEFORE AND AFTER POSTING OF SPEED LIMITS (NON-LOCAL VEHICLES)

FIGURE 15



85TH PERCENTILE SPEED BEFORE AND AFTER
POSTING OF SPEED LIMITS
(DAY CONDITION)

FIGURE 16



85TH PERCENTILE SPEED BEFORE AND AFTER
POSTING OF SPEED LIMITS
(NIGHT CONDITION)

FIGURE 17

night conditions the 50 mile per hour speed limit is slightly above the 85th percentile speed.

Location A

Station A 1

It should be noticed that the within cell variation is considerable for the night condition before the "Speed Zone Ahead" sign was posted. It is difficult to explain these differences because both studies were conducted for the same weather and surface conditions. However, one study was conducted in April, the other one in August. Seasonal variations in the speed pattern may partially contribute to this great difference but it should also be noticed that this difference is not pronounced for the day condition, nor for local and non-local vehicles. The effect of the "Speed Zone Ahead" sign is thus negligible. This is probably due to the fact that the 85th percentile speed is considerably lower than the state-wide limit caused by a natural reduction of speed when passing the intersection with U.S. 52 Bypass (see Figure 2).

Station A 2

The results of the studies at this station are presented in Table 11. There is a significantly lower speed at night than during the day. The appearance of the illuminated city ahead may involuntarily cause a reduction of speed. No significant effect of posting a 50 mile per hour speed limit sign was registered.

Station A 3

This station is the first one within the developed area and was

assigned a 40 mile per hour speed limit. Although this station was used as a control station where weekly surveys were conducted, no significant change was recorded due to sign condition. Non-local vehicles traveled noticeably faster than local vehicles; however, no difference was found between day and night. Between stations A 3 and A 4 there is a crosswalk used by school children at certain times during the day. A particular study at this crosswalk (31) disclosed that the vehicles significantly reduced their speed when children were present, but not otherwise. At station A 3 a specific speed limit sign, "SPEED LIMIT 25 MPH WHEN CHILDREN PRESENT" was placed. Speed studies at station A 3 recorded no change in traveled speed, probably due to the fact that the crosswalk is barely visible from the sign. But the lack of change in speed also indicates no respect for the sign by itself.

Station A 4

Non-local vehicles were still traveling significantly faster than local ones within the 40 mile per hour speed zone. A significant change in the mean speed because of posting the sign was recorded but surprisingly in the opposite direction. The mean speed was higher after than prior to the posting of a speed limit. A noticeable difference between day and night 85th percentile speed was also disclosed at this station.

Station A 5

After posting the 40 mile per hour speed limit the mean speed was significantly higher but not the 85th percentile speed. Non-local vehicles still traveled considerably faster than local ones.

Station A 6

Customarily, reduction of speed limits is posted by increments

of ten miles per hour. An exception was made at this station where a 35 mile per hour speed limit sign was posted, thus reducing the speed limit by only five miles per hour, in order to better coincide with the actual 85th percentile speed. No significant change in traveled speeds after the posting of the sign was revealed. At night, however, the 85th percentile speed was conspicuously lower than during daytime. Six months after the entire program of speed surveys were completed a check survey was conducted at this station and revealed a slight reduction, but still insignificant, of the traveled speed. The 35 mile per hour speed limit sign was then replaced by a 30 mile per hour speed limit sign, and the results are shown in Table 12. It is obvious from the figures, and a statistical analysis also verifies it, that the lowering of the speed limit from 35 miles per hour to 30 miles per hour did not have any effect whatsoever.

Station A 7

The location of this station immediately ahead of a sharp curve (see Figures 2 and 4) gives as a result a speed pattern with very little variation. Most vehicles brake before entering the curve thus reducing their speed in order to achieve comfortable riding through the curve. None of the factors exhibited any significant change because of posting a 30 mile per hour speed limit.

Stations A 11, A 14, A 16, A 17

Speed studies in the outbound direction were not originally planned but were added later. For this reason no data as to vehicle residence were available when no signs existed. The study in the outbound direction, essentially characterized as a check on directional

TABLE 12

OBSERVED RESULTS ON SPEED OF LOWERING A SPEED LIMIT

Passenger Vehicles	Speed Limit	
	35 mph	30 mph
Mean Speed	33.91	34.31
85th Percentile Speed	38.42	38.49
Speed Differential	9.09	8.47
10 mph Pace	28-37	30-39
Per Cent in Pace	74.6	78.9
Standard Deviation	4.70	4.26
Per Cent of Vehicles exceeding the speed limit by 5 mph or more	10.1	37.8

speed patterns, revealed a significantly lower speed at station A 16 after a 35 mile per hour speed limit had been established. For any other sign condition no noticeable change was recorded. An analysis of differences in directional speeds was regarded of little importance because a speed zone in one direction very seldom coincides with a similar zone in the other direction.

Location B

This location exhibited specific characteristics because of the absence of extensive development along the highway. For this reason it could not advantageously be compared with and analyzed together with

any other location. Although 65 miles per hour is permitted the 85th percentile speed was 55.9 miles per hour in the open area and 54.0 miles per hour at the "Speed Zone Ahead" sign during the day. A relatively sharp curve immediately beyond the 40 mile per hour speed limit sign reduced the 85th percentile speed to 30.5 miles per hour (63). The 50 mile per hour speed zone, about three miles long, revealed a natural reduction of the 85th percentile speed from 53.3 miles per hour at the beginning of the speed zone to 50.6 miles per hour at the end of the same zone. During the night, vehicles reduced their speed considerably more within the 50 mile per hour zone than during the day. The absence of illumination along the road, the darkness because of vegetation, and the non-existence of development may explain this phenomena. Furthermore, non-local vehicles also reduced their speed substantially more than local vehicles which obviously are more familiar with the road.

Locations C, D, and E

These three locations, each one displaying a predominant type of development, were statistically tested against each other in order to determine whether or not there is any significant difference in speed pattern by type of development. The result, disclosed in Table 11, revealed significant differences among most of the factors, including even significant interaction. This may indicate that certain combinations of development, sign condition, and other factors significantly change the speed pattern. The lack of comparability of the locations may explain the significant interactions. It was generally found that non-local vehicles at all locations and for all sign conditions traveled

noticeably faster than local vehicles. Night driving characteristics differed from day driving characteristics in many respects depending upon location and sign condition. Illumination of the highway together with alignment attributes may also have played a significant role.

CONCLUSIONS AND RECOMMENDATIONS

This study has revealed that drivers, in general, do not drive according to posted speed limit signs. Most of them select a speed which they consider proper, reasonable, and safe for conditions prevalent, regardless of regulations. Moreover, it is shown that when speed limits are determined from the 85th percentile speed, traveled speeds are not affected. The fear that the establishment of a speed limit which might appear relatively high will create a new speed pattern with faster speeds appears to have no foundation.

The study of the effect of roadside development on speed patterns did not produce conclusive results, but the many significant differences noted in the speed patterns indicate a possibility of an effect on speed pattern by type of development.

Some authors (85) have expressed doubts concerning the reliability of fixed-point speed measurements. Hourly, daily, and monthly variations of the speed pattern constitute an enigma which needs to be investigated more carefully. The classical method of using the number of observations as the sole criteria for determining the adequacy of a sample may give erroneous or misleading results. Even in this study unexplained variations in speed occurred under the very same sign conditions, keeping other factors presumedly as constant as possible. Selected locations at which speed studies could be conducted continuously would probably

increase the knowledge on seasonal variations of speed. Then, if necessary, correction factors could be developed which could be used to eliminate the bias that occurs because of hourly, daily, and monthly variation of speeds. It is recommended that such a study be initiated.

It is undoubtedly evident that the relationship between speed and roadside development is extremely complex. More detailed studies taking into account all of the factors, many of which were not tested in this study, will probably increase the knowledge and eventually develop additional warrants for establishing reasonable speed limits.

Most drivers will obey a posted speed limit if, and only if, the sign is properly posted and the numerical limit is reasonable. Such signing also permits a more efficient enforcement of speed limits, whereas enforcement of unreasonable speed limits efficiently requires a police force which no community can afford.

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APPENDIX A

NOTATION USED

A	Speed study location A	
	A 1	Speed measurement station 1 at location A
	A 2	Speed measurement station 2 at location A
α	Level of significance	
B	Speed study location B	
β	Position angle of radar unit with road edge	
C	Speed study location C	
c	Speed limit sign condition	
	c 1	Sign condition 1 (see page 90)
	c 2	Sign condition 2 (see page 90)
D	Speed study location D	
d	Traffic flow direction	
	d 1	Inbound flow
	d 2	Outbound flow
df	Degrees of freedom	
E	Speed study location E	
e	Error	
F	Function of the F distribution	
f	Frequency of vehicles for a specific speed	
g	Number of replications for the same sign condition (cell)	
H	Statistical hypothesis	

i	Index of summation
j	Index of summation
k	Number of conditions (cells)
L	Location
MS	Mean square
MSe	Error mean square
m	Number of vehicles exceeding a posted speed limit by five miles per hour or more
N	Number of vehicles (volume) passing a speed measurement station
n	Number of vehicles, the speed of which is recorded, passing a speed measurement station $(n < N)$
R	Range
S	Effective range in feet of radar speed meter
SS	Sum of squares
SZA	Speed Zone Ahead
s	Standard deviation for a sample statistic
t	Time factor
$t\ 1$	Day
$t\ 2$	Night
u	Transformed fraction (see page 57)
V	Calculated speed from a frequency distribution
$\overline{V}$	Mean speed
V_{15}	15th percentile speed
V_{50}	50th percentile (median) speed
V_{85}	85th percentile speed
V_{diff}	Speed differential

v	Vehicle factor	
	v 1	Local vehicles
	v 2	Non-local vehicles
w	Lane width, roadway width, in feet	
X	Variable in a frequency distribution, e. g., the speed	
x	Variable in a frequency distribution, e. g., the speed	
y	Lateral distance in feet from radar unit to road edge	
z	Fraction of vehicles exceeding a posted speed limit by five miles per hour or more (see also page 57)	

DEFINITION OF STATISTICAL TERMS

General Definitions

Population	The total group of elements under investigation
Sample	A number of elements selected from the population
Normal Distribution	A certain theoretical relation between values of the measured variable and the relative frequency of the value
Population Statistic	A numerical characteristic of the population, e. g., the population mean
Sample Statistic	A numerical characteristic of the sample, e. g., the sample mean
Variance	A measure of the variability or dispersion of the values
Standard Deviation	Square root of the variance
Statistical Hypothesis	A statement delimiting the possible value that a population parameter may have

Definitions Related to Speed

Mean Speed	The average of individual speeds; $\overline{V} = \frac{\sum_{i=1}^{n} V_i}{n}$ where V_i is the individual speed measurement. For data arranged in a frequency distribution containing speed classes and frequencies within these classes $\overline{V} = \frac{\sum_{i=1}^{n} f_i V_i}{\sum_{i=1}^{n} f_i}$ where V_i now is the midvalue of class i and f_i the frequency of measurements in class i. By definition $n = \sum_{i=1}^{n} f_i$
pth Percentile Speed	That speed at and below which p per cent of the drivers travel
Median Speed	The 50th percentile speed
Speed Differential	The difference between the 85th percentile speed and the 15th percentile speed; $V_{diff} = V_{85} - V_{15}$
10 mph Pace	The speed increment of 10 miles per hour in which most of the drivers are included
Per Cent in Pace	Per cent of drivers within the pace limits

Range	The difference between the fastest and the slowest speed recorded for a particular condition; $R = V_{max} - V_{min}$
Variance	A measure of the variability or dispersion of the speed within a speed distribution, defined by $s^2 = \frac{\sum_{i=1}^{n} f_i (V_i - \bar{V})^2}{n - 1}$
Standard Deviation	Square root of the variance $S = \sqrt{\frac{\sum_{i=1}^{n} f_i (V_i - \bar{V})^2}{n - 1}}$
Mean Square	Sum of squares divided by degrees of freedom

SPEED LIMIT SIGN CONDITIONS

The following notation is consistently used to delineate the various sign conditions at location A. The signs were posted in the same order as the numbering outlines.

c 1	No speed limit sign posted at any station.
c 2	40 mile per hour speed limit signs posted, in inbound direction, north of station A 3 and A 5; in outbound direction, opposite station A 5.
c 3	Additional 40 mile per hour speed limit signs posted north of station A 4 in inbound direction; and at station A 14 in outbound direction.
c 4	35 mile per hour signs posted north of inbound station A 6 and at outbound station A 16.
c 5	30 mile per hour speed limit signs posted at inbound station A 7 and south of outbound station A 17.
c 6	50 mile per hour speed limit signs posted at inbound station A 2 and opposite this station for the outbound direction.
c 7	"Speed Zone Ahead" sign posted at inbound station A 1. "Speed Zone Ends" sign posted in outbound direction at the intersection with U.S. 52 Bypass.

c 8 Special signs with inscription "SPEED LIMIT 25 MPH WHEN CHILDREN PRESENT" posted at inbound station A 3 and outbound station A 14.

c 9 The 35 mile per hour sign north of inbound station A 6 replaced by a 30 mile per hour speed limit sign; the 30 mile per hour sign at station A 7 simultaneously removed.

For additional reference, see Figure 2.

At the other locations the numerical limit is used as a notation for the sign condition because the limits were established prior to the studies.

APPENDIX B

SUMMARY OF DATA

In order to keep this report within reasonable size publishing of the numerous speed distributions obtained is deliberately omitted. The most important data calculated for various conditions and factors are compiled in Table 13, which concerns speed parameters, and Table 14, which deals with speed violations. Notations used in those tables are defined in Appendix A.

TABLE 13

SUMMARY OF DATA: SPEED PARAMETERS

LOCATION STATION	FACTOR	SIGN COND.	N	n	MEAN SPEED	SPEED PERCENTILES			SPEED DIFF.	10 MPH PACE		STD. DEV.
						V_{15}	V_{50}	V_{85}		RANGE	PER CENT	
A 1	v 1	c 1	88	80	43.05	35.97	42.50	51.09	15.12	36–45	52.6	7.35
			78	70	43.81	35.64	43.27	52.27	16.63	40–49	50.0	7.41
			91	74	44.53	37.80	43.90	52.63	14.83	40–49	51.3	7.15
		c 7	67	60	42.27	33.98	41.50	50.84	16.86	34–43	46.6	7.66
	v 2	c 1	97	88	45.66	37.61	45.52	52.41	14.80	42–51	54.6	7.35
			111	90	43.25	33.66	42.58	50.78	17.12	38–47	55.6	7.69
			96	89	44.28	38.57	44.19	50.41	11.84	40–49	61.7	5.93
		c 7	104	95	43.24	36.53	42.68	50.47	13.94	36–45	58.8	6.41
	t 1	c 1	185	168	44.42	36.46	44.35	51.79	15.33	40–49	48.7	7.45
			189	160	43.50	35.53	42.86	51.15	15.62	38–47	50.2	7.57
			187	163	44.39	38.22	44.07	51.42	13.20	40–49	57.0	6.50
		c 7	171	155	42.86	35.84	42.06	50.57	14.73	36–45	53.1	6.95
	t 2	c 1	111	101	44.35	36.13	43.19	53.63	17.50	36–45	52.6	8.08
			162	143	38.63	30.93	38.85	45.28	14.35	36–45	55.9	6.72
		c 7	155	138	38.49	30.38	38.07	46.32	15.94	32–41	52.9	7.76

TABLE 13 (continued)

SUMMARY OF DATA: SPEED PARAMETERS

LOCATION STATION	FACTOR	SIGN COND.	N	n	MEAN SPEED	SPEED PERCENTILES			SPEED DIFF.	10 MPH PACE		STD. DEV.
						V_{15}	V_{50}	V_{85}		RANGE	PER CENT	
A 2	v 1	c 1	90	87	42.63	36.69	43.13	51.71	15.02	38-47	52.8	6.72
			72	60	42.77	36.68	42.49	49.19	12.51	38-47	56.7	5.84
			75	68	44.38	37.91	44.51	51.36	13.45	38-47	60.3	6.22
		c 6	89	77	44.42	37.65	43.59	50.93	13.28	36-45	58.5	6.27
		c 7	85	74	44.39	37.99	44.74	51.22	13.23	40-49	56.7	6.72
			73	59	42.57	35.83	42.50	49.97	14.14	36-45	60.9	5.96
			70	60	46.20	39.21	45.77	53.50	14.29	38-47	55.1	7.06
			87	72	47.22	39.41	47.77	55.57	16.16	42-51	54.1	6.90
	v 2	c 1	94	93	43.33	37.97	44.50	50.41	12.44	40-49	62.3	5.52
			109	96	45.36	39.55	44.87	52.31	12.76	40-49	60.4	6.13
			113	101	47.25	39.95	46.94	54.95	15.00	42-51	58.4	7.15
		c 6	114	92	44.98	38.34	44.96	50.78	12.44	38-47	57.5	6.35
		c 7	124	107	47.32	41.53	47.00	53.88	12.35	42-51	61.7	5.86
			134	112	45.04	39.97	44.84	50.62	10.65	42-51	66.9	5.46
			100	85	46.76	39.69	46.50	53.63	13.94	42-51	54.0	6.82
			137	112	47.87	41.97	46.50	53.86	11.89	44-53	60.7	5.46

TABLE 13 (continued)

SUMMARY OF DATA: SPEED PARAMETERS

LOCATION STATION	FACTOR	SIGN COND.	N	n	MEAN SPEED	SPEED PERCENTILES			SPEED DIFF.	10 MPH PACE		STD. DEV.
						V_{15}	V_{50}	V_{85}		RANGE	PER CENT	
A 2	t 1	c 1	184	180	43.99	37.29	44.01	50.95	13.66	40-49	56.2	6.13
			181	156	44.36	38.03	44.20	50.83	12.80	40-49	58.4	6.15
			188	169	46.10	38.93	45.99	53.70	14.77	42-51	57.4	6.92
		c 6	203	169	44.73	38.05	44.31	50.84	12.79	38-47	56.2	6.32
		c 7	209	181	46.12	40.10	46.07	53.14	13.04	42-51	58.7	6.40
			207	171	44.19	37.93	44.23	50.41	12.48	38-47	62.7	5.76
			170	145	46.78	39.47	46.26	53.56	14.09	40-49	53.2	6.92
			224	184	47.62	41.09	47.62	54.37	13.28	42-51	58.2	6.06
	t 2	c 1	200	195	42.69	35.75	43.55	51.00	15.25	38-47	52.9	7.36
			205	165	39.65	32.12	39.27	46.56	14.44	32-41	52.7	6.92
			244	208	41.79	34.46	40.77	49.98	15.52	34-43	55.8	7.31
		c 6	173	147	41.33	33.21	40.86	49.69	16.48	36-45	52.4	7.15
		c 7	156	138	40.80	34.63	40.50	46.54	11.91	36-45	61.7	5.91
			140	126	43.87	37.47	42.92	51.06	13.59	40-49	57.8	6.24
			110	97	44.15	36.89	43.99	50.75	13.86	40-49	54.7	4.26
			112	102	43.70	36.68	42.96	50.57	13.89	40-49	58.8	6.36

TABLE 13 (continued)

SUMMARY OF DATA: SPEED PARAMETERS

LOCATION STATION	FACTOR	SIGN COND.	N	n	MEAN SPEED	SPEED PERCENTILES			SPEED DIFF.	10 MPH PACE		STD. DEV.
						V_{15}	V_{50}	V_{85}		RANGE	PER CENT	
A 3	v 1	c 1	91	87	35.08	28.90	34.38	41.53	12.63	28-37	61.9	6.02
			123	110	34.06	29.00	33.64	39.94	10.94	28-37	73.8	4.88
		c 2	116	101	34.68	29.53	34.94	38.81	9.28	30-39	75.3	4.90
			109	85	35.23	30.98	34.91	39.61	8.63	30-39	79.9	4.36
			115	100	36.28	31.50	37.08	42.83	11.33	32-41	64.0	5.65
			95	80	35.65	30.85	35.62	40.67	9.82	30-39	73.9	4.89
			100	86	35.80	30.20	35.94	41.52	11.32	30-39	68.6	4.84
			110	103	36.15	30.83	35.27	41.76	10.93	30-39	66.1	5.31
		c 6	109	88	35.55	30.34	35.62	40.33	9.99	30-39	70.5	4.68
		c 7	88	78	34.55	29.67	34.62	38.72	9.05	30-39	74.4	4.68
			144	115	35.56	30.07	35.37	41.77	11.70	30-39	65.3	4.92
			107	87	34.15	29.15	34.59	39.68	10.53	30-39	67.9	4.86
			75	63	37.77	31.60	37.05	43.03	11.43	34-43	69.7	5.10
			107	84	36.17	30.35	36.17	42.11	11.76	34-43	64.2	5.54

TABLE 13 (continued)

SUMMARY OF DATA: SPEED PARAMETERS

LOCATION STATION	FACTOR	SIGN COND.	N	n	MEAN SPEED	SPEED PERCENTILES			SPEED DIFF.	10 MPH PACE		STD. DEV.
						V_{15}	V_{50}	V_{85}		RANGE	PER CENT	
A 3	v 2	c 1	65	64	37.63	32.37	37.50	43.33	10.96	32-41	65.6	5.08
			90	83	37.50	31.00	36.85	44.06	13.06	32-41	59.2	6.64
		c 2	100	84	37.62	31.90	37.95	43.35	11.45	32-41	63.1	5.34
			125	104	37.23	32.75	37.33	41.98	9.23	32-41	74.9	4.26
			96	82	37.11	32.73	37.00	41.08	8.35	32-41	78.1	4.33
			124	104	37.67	31.74	37.15	42.59	10.85	34-43	69.2	5.89
			119	102	36.54	31.97	36.83	42.83	10.86	32-41	71.6	5.65
			111	97	36.67	30.94	36.71	42.48	11.54	30-39	62.9	5.34
		c 6	127	98	37.42	32.85	37.08	41.41	8.56	32-41	75.6	5.08
		c 7	112	95	36.37	32.00	35.92	41.10	9.10	32-41	75.7	4.41
			111	98	36.77	31.40	36.67	42.06	10.66	34-43	72.5	4.97
			127	103	36.46	31.17	36.44	41.59	10.42	34-43	70.9	4.72
			110	98	38.46	32.08	38.28	44.79	12.71	32-41	60.3	6.08
			97	81	37.29	32.99	37.03	41.86	8.87	32-41	75.3	4.94

TABLE 13 (continued)

SUMMARY OF DATA: SPEED PARAMETERS

LOCATION STATION	FACTOR	SIGN COND.	N	n	MEAN SPEED	SPEED PERCENTILES			SPEED DIFF.	10 MPH PACE		STD. DEV.
						V_{15}	V_{50}	V_{85}		RANGE	PER CENT	
A 3	t 1	c 1	156	151	36.15	30.02	35.58	42.48	12.46	32-41	60.9	5.65
			213	193	35.54	29.58	34.89	41.40	11.82	28-37	64.3	5.28
		c 2	216	185	36.01	30.61	35.91	41.72	11.11	32-41	65.9	5.31
			234	189	36.33	31.63	36.02	41.10	9.37	30-39	74.6	4.41
			211	182	37.20	32.05	37.04	42.05	10.00	32-41	70.3	5.08
			219	184	36.79	31.22	36.50	41.90	10.68	32-41	67.8	5.56
			219	188	36.74	31.59	36.42	41.94	10.35	32-41	69.2	5.37
			221	200	36.40	30.38	36.07	42.30	11.47	30-39	64.5	5.33
		c 6	236	186	36.53	31.35	36.41	41.03	9.68	32-41	72.7	4.98
		c 7	200	173	35.55	30.97	35.35	40.37	9.40	30-39	72.9	4.62
			255	213	36.12	30.62	35.95	41.89	11.27	34-43	58.5	4.98
			234	190	35.41	30.09	35.52	40.66	10.57	32-41	66.9	4.92
			185	161	38.19	32.26	37.69	43.98	11.72	34-43	63.4	5.74
			204	165	36.72	31.74	36.69	41.98	10.24	32-41	69.1	5.28

TABLE 13 (continued)

SUMMARY OF DATA: SPEED PARAMETERS

LOCATION STATION	FACTOR	SIGN COND.	N	n	MEAN SPEED	SPEED PERCENTILES			SPEED DIFF.	10 MPH PACE		STD. DEV.
						V_{15}	V_{50}	V_{85}		RANGE	PER CENT	
A 3	t 2	c 1	129	124	36.53	29.22	35.50	42.97	13.75	32-41	59.6	6.64
			216	203	36.09	30.38	35.48	41.41	11.03	32-41	63.5	5.36
		c 2	210	168	35.49	30.51	34.79	40.85	10.34	30-39	68.4	5.04
			171	146	36.61	30.87	36.17	42.35	11.48	32-41	63.7	5.28
			159	133	36.24	30.30	35.33	41.50	11.20	32-41	64.6	6.00
			204	169	35.74	31.02	35.40	40.40	9.38	30-39	74.5	4.99
			164	146	36.69	30.65	36.54	43.15	12.50	32-41	60.3	5.62
			151	123	34.97	29.71	34.75	40.51	10.80	30-39	68.3	5.32
		c 6	238	195	35.61	30.24	35.14	41.75	11.51	30-39	65.7	5.34
		c 7	242	191	35.24	29.80	34.91	40.79	10.99	30-39	57.1	5.31
			173	146	34.76	29.13	34.36	40.70	11.57	30-39	63.7	5.42
			172	145	34.87	28.72	34.43	40.95	12.23	32-41	61.4	5.88
			148	126	37.86	31.50	36.98	42.52	11.02	32-41	64.3	5.50
			112	101	38.36	31.68	38.38	45.02	13.34	32-41	53.4	6.46

TABLE 13 (continued)

SUMMARY OF DATA: SPEED PARAMETERS

LOCATION STATION	FACTOR	SIGN COND.	N	n	MEAN SPEED	SPEED PERCENTILES			SPEED DIFF.	10 MPH PACE		STD. DEV.
						V_{15}	V_{50}	V_{85}		RANGE	PER CENT	
A 4	v 1	c 1	122	116	33.72	28.00	33.50	39.42	11.42	28-37	73.0	5.72
		c 2	138	107	36.56	31.50	36.18	40.77	9.27	32-41	74.8	4.59
		c 3	93	79	36.20	31.22	35.96	41.80	9.58	32-41	67.1	5.42
			111	92	36.43	31.13	36.35	41.32	10.19	30-39	71.7	4.86
			134	110	35.79	29.59	35.29	40.68	11.09	30-39	71.8	4.62
		c 7	139	109	35.86	30.29	35.34	41.58	11.29	32-41	52.4	5.19
	v 2	c 1	94	90	36.23	30.48	35.64	43.01	12.53	28-37	62.2	5.80
		c 2	85	76	37.92	33.92	38.01	43.67	9.75	32-41	75.0	4.83
		c 3	101	85	38.03	32.84	37.82	42.90	10.06	32-41	73.0	4.74
			125	110	37.86	32.52	37.50	43.98	11.46	32-41	70.1	5.64
			115	103	36.99	31.57	37.20	41.43	9.86	32-41	73.8	4.70
		c 7	138	117	37.61	31.63	37.35	42.99	11.36	32-41	71.7	4.96

TABLE 13 (continued)

SUMMARY OF DATA: SPEED PARAMETERS

LOCATION	FACTOR	SIGN	N	n	MEAN	SPEED PERCENTILES			SPEED	10 MPH PACE		STD.
STATION		COND.			SPEED	V_{15}	V_{50}	V_{85}	DIFF.	RANGE	PER CENT	DEV.
A 4	t 1	c 1	216	206	34.82	28.83	34.40	41.91	13.08	28-37	62.7	5.88
		c 2	223	183	37.30	32.59	36.85	41.39	8.80	32-41	75.0	4.78
		c 3	194	164	37.23	32.14	36.94	42.25	10.11	32-41	70.3	5.04
			236	202	37.21	31.84	36.99	42.64	10.80	32-41	69.3	5.34
			249	213	36.37	31.50	36.22	41.06	9.56	32-41	72.4	4.70
		c 7	277	226	36.77	31.50	36.67	42.05	10.55	32-41	67.7	5.08
	t 2	c 1	148	146	36.17	29.98	35.98	43.00	13.02	30-39	59.5	6.12
		c 2	197	169	37.14	30.91	36.94	43.03	12.12	32-41	62.7	5.94
		c 3	228	187	36.42	30.91	36.00	42.35	10.44	32-41	65.3	5.81
			149	129	37.28	31.78	37.05	42.51	10.73	32-41	66.0	5.29
		c 7	169	158	36.73	32.10	36.48	42.72	10.62	32-41	65.5	5.70

TABLE 13 (continued)

SUMMARY OF DATA: SPEED PARAMETERS

LOCATION STATION	FACTOR	SIGN COND.	N	n	MEAN SPEED	SPEED PERCENTILES			SPEED DIFF.	10 MPH PACE		STD. DEV.
						V_{15}	V_{50}	V_{85}		RANGE	PER CENT	
A 5	v 1	c 1	114	103	32.70	28.93	33.00	39.16	10.23	28-37	73.8	4.75
		c 2	173	147	35.51	30.85	35.25	40.12	9.27	30-39	74.8	4.59
		c 3	161	137	36.70	32.21	36.62	41.01	8.80	32-41	76.0	4.41
			126	104	36.63	32.65	36.28	40.72	8.07	32-41	81.7	3.95
		c 7	139	115	35.16	29.55	35.44	40.47	10.92	30-39	66.1	5.07
	v 2	c 1	83	79	35.96	32.63	36.05	41.84	9.21	32-41	77.2	4.84
		c 2	118	93	38.56	33.93	38.70	43.25	9.32	34-43	73.2	4.71
		c 3	136	118	38.92	34.13	38.72	44.16	10.03	34-43	71.2	4.44
			102	82	38.48	33.74	39.58	42.77	9.03	34-43	75.6	4.27
		c 7	105	85	36.97	32.54	36.77	41.05	8.51	32-41	79.9	4.31

TABLE 13 (continued)

SUMMARY OF DATA: SPEED PARAMETERS

LOCATION STATION	FACTOR	SIGN COND.	N	n	MEAN SPEED	SPEED PERCENTILES			SPEED DIFF.	10 MPH PACE		STD. DEV.
						V_{15}	V_{50}	V_{85}		RANGE	PER CENT	
A 5	t 1	c 1	197	182	35.12	29.94	34.53	40.52	10.58	28-37	69.2	5.05
		c 2	291	240	36.69	31.71	36.46	41.50	9.79	32-41	71.3	4.88
		c 3	297	255	37.73	33.30	37.60	42.64	9.79	34-43	72.5	4.56
			228	186	37.45	33.04	37.07	41.77	8.73	32-41	78.0	4.20
		c 7	244	200	35.93	30.68	36.06	40.77	10.09	32-41	70.5	4.84
	t 2	c 1	211	195	33.75	29.52	34.37	40.45	10.93	30-39	66.2	5.04
		c 2	237	206	36.18	31.68	35.96	41.09	9.41	32-41	73.3	4.50
		c 3	239	195	35.65	29.60	35.67	40.57	10.97	30-39	70.3	4.75
		c 7	221	182	34.98	29.98	34.43	39.78	9.80	30-39	72.0	5.01

TABLE 13 (continued)

SUMMARY OF DATA: SPEED PARAMETERS

LOCATION STATION	FACTOR	SIGN COND.	N	n	MEAN SPEED	SPEED PERCENTILES			SPEED DIFF.	10 MPH PACE		STD. DEV.
						V_{15}	V_{50}	V_{85}		RANGE	PER CENT	
A 6	v 1	c 1	155	143	32.28	27.90	31.84	37.61	9.71	26-35	77.6	4.24
		c 3	175	129	33.03	28.62	33.06	37.28	8.66	28-37	77.6	4.00
		c 5	165	126	32.64	28.45	32.64	36.78	8.33	28-37	84.2	3.88
		c 6	202	143	32.36	27.99	31.98	37.23	9.24	28-37	74.1	4.44
		c 7	158	118	32.40	28.17	32.19	36.64	8.47	28-37	79.7	4.23
			146	116	33.64	29.62	33.66	38.20	8.58	28-37	78.6	4.28
			148	114	35.26	30.23	35.07	40.05	9.72	30-39	71.1	4.94
	v 2	c 1	122	116	31.64	26.60	31.11	37.17	10.57	28-37	68.2	5.08
		c 3	168	121	33.85	28.59	33.74	39.43	10.84	28-37	67.9	5.26
		c 5	116	93	33.68	28.78	32.93	38.97	10.19	28-37	74.1	5.31
		c 6	138	115	32.73	29.52	32.51	36.70	7.18	28-37	80.9	4.13
		c 7	117	92	32.96	28.44	32.66	38.05	9.61	30-39	71.7	4.98
			122	100	35.10	30.43	34.38	40.17	9.74	30-39	74.0	5.72
			118	103	35.36	31.41	35.37	39.44	8.03	30-39	81.5	3.68

TABLE 13 (continued)

SUMMARY OF DATA: SPEED PARAMETERS

LOCATION STATION	FACTOR	SIGN COND.	N	n	MEAN SPEED	SPEED PERCENTILES			SPEED DIFF.	10 MPH PACE		STD. DEV.
						V_{15}	V_{50}	V_{85}		RANGE	PER CENT	
A 6	t 1	c 1	277	259	31.99	27.61	31.53	37.38	9.77	26-35	73.0	4.65
		c 3	343	250	33.48	28.61	33.37	38.44	9.83	28-37	72.8	4.83
		c 5	281	219	33.08	28.58	32.75	37.34	8.76	28-37	79.8	4.58
		c 6	340	258	32.52	27.60	32.26	37.02	9.42	28-37	77.1	4.31
		c 7	275	210	32.64	28.25	32.41	37.26	9.01	28-37	76.1	5.58
			268	216	34.31	29.94	33.99	39.19	9.25	30-39	75.5	4.48
			266	217	35.30	30.77	35.23	39.76	8.99	30-39	76.1	4.38
	t 2	c 1	195	186	31.61	26.36	31.78	36.78	10.36	26-35	69.9	4.92
		c 3	231	175	31.72	26.53	31.32	36.84	10.31	28-37	69.7	4.98
		c 5	277	217	30.35	26.10	30.17	34.30	8.20	26-35	76.3	4.25
		c 6	280	211	30.79	26.61	30.59	34.84	8.23	26-35	78.6	4.12
		c 7	184	147	34.64	30.23	34.42	39.02	8.79	30-39	77.6	4.34
			143	119	33.24	29.37	33.21	37.42	8.05	28-37	80.8	4.68
			163	141	33.56	29.33	33.33	37.78	8.45	30-39	76.7	4.26

TABLE 13 (continued)

SUMMARY OF DATA: SPEED PARAMETERS

LOCATION STATION	FACTOR	SIGN COND.	N	n	MEAN SPEED	SPEED PERCENTILES			SPEED DIFF.	10 MPH PACE		STD. DEV.
						V_{15}	V_{50}	V_{85}		RANGE	PER CENT	
A 7	v 1	c 1	162	155	26.97	22.63	27.00	31.07	8.44	22-31	79.5	4.20
		c 2	183	145	28.42	24.44	28.02	32.54	8.10	24-33	82.1	3.78
		c 3	169	142	28.18	24.45	27.68	31.91	7.46	24-33	86.0	3.83
		c 4	188	144	28.10	24.62	27.71	31.46	6.84	24-33	83.9	3.69
			152	119	28.20	23.60	27.93	32.04	8.44	24-33	87.5	3.55
		c 7	167	133	28.61	25.21	28.50	31.97	6.76	24-33	86.5	3.26
	v 2	c 1	86	80	27.77	23.85	27.98	31.00	7.15	22-31	83.8	3.92
		c 2	113	91	27.44	73.20	27.95	31.10	7.90	22-31	82.4	3.73
		c 3	127	100	27.70	24.24	27.03	31.79	7.55	24-33	83.0	3.63
		c 4	120	90	27.30	22.91	27.23	30.78	7.87	22-31	83.4	4.22
			124	99	28.22	24.08	28.32	32.01	7.93	24-33	82.8	4.20
		c 7	172	129	28.67	25.01	28.69	32.26	7.25	24-33	88.3	3.24

TABLE 13 (continued)

SUMMARY OF DATA: SPEED PARAMETERS

LOCATION STATION	FACTOR	SIGN COND.	N	n	MEAN SPEED	SPEED PERCENTILES			SPEED DIFF.	10 MPH PACE		STD. DEV.
						V_{15}	V_{50}	V_{85}		RANGE	PER CENT	
A 7	t 1	c 1	248	235	27.24	22.91	27.42	31.06	8.15	22-31	80.8	4.12
		c 2	296	236	28.04	24.06	27.96	31.76	7.70	24-33	81.0	3.79
		c 3	296	242	27.98	24.36	27.34	31.86	7.50	24-33	84.8	3.76
		c 4	308	234	27.79	24.08	27.54	31.27	7.19	22-31	82.5	3.91
			276	218	28.21	24.32	28.13	32.03	7.71	24-33	85.3	3.86
		c 7	339	262	28.64	25.09	28.59	32.12	6.88	24-33	87.4	3.26
	t 2	c 1	214	201	28.08	23.23	28.06	33.23	10.00	24-33	70.1	4.56
		c 2	273	220	26.82	22.19	26.67	31.15	8.96	22-31	76.7	4.42
		c 3	267	220	27.37	23.19	27.19	31.84	8.65	22-31	76.3	4.08
		c 4	235	201	27.52	22.91	27.24	32.22	9.31	24-33	73.5	4.32
		c 7	189	170	29.56	24.99	29.32	33.59	8.40	24-33	75.8	4.62

TABLE 13 (continued)

SUMMARY OF DATA: SPEED PARAMETERS

LOCATION STATION	FACTOR	SIGN COND.	N	n	MEAN SPEED	SPEED PERCENTILES			SPEED DIFF.	10 MPH PACE		STD. DEV.
						V_{15}	V_{50}	V_{85}		RANGE	PER CENT	
A 11	v 1	c 5	62	55	43.59	36.43	43.08	51.35	14.92	36-45	54.5	6.48
		c 6	48	45	45.21	39.81	44.88	49.77	9.96	40-49	71.1	6.04
			79	67	44.14	38.31	44.50	49.05	10.74	40-49	68.6	5.59
	v 2	c 5	72	56	44.14	37.97	42.40	51.37	13.40	36-45	64.2	7.25
		c 6	80	68	45.20	39.99	44.58	51.43	11.44	40-49	66.2	5.72
			86	73	43.05	36.99	42.79	49.78	12.79	38-47	57.5	6.19
	t 1	c 1	103	85	44.72	37.64	44.21	53.59	15.95	38-47	54.1	6.96
		c 5	134	111	43.87	37.03	42.69	51.35	14.32	36-45	59.5	6.88
		c 6	128	113	45.21	39.93	44.69	51.02	11.09	40-49	68.0	5.86
			165	141	43.57	37.64	44.07	49.38	11.74	40-49	61.4	5.94

TABLE 13 (continued)

SUMMARY OF DATA: SPEED PARAMETERS

LOCATION STATION	FACTOR	SIGN COND.	N	n	MEAN SPEED	SPEED PERCENTILES			SPEED DIFF.	10 MPH PACE		STD. DEV.
						V_{15}	V_{50}	V_{85}		RANGE	PER CENT	
A 14	v 1	c 2	145	113	37.30	33.50	37.17	41.66	8.16	34-43	80.6	3.89
			127	102	37.26	32.63	37.59	42.09	9.46	32-41	74.6	4.39
		c 3	94	84	38.36	33.59	38.13	42.72	9.13	34-43	75.0	4.73
		c 4	93	77	38.86	34.43	38.26	43.32	8.89	34-43	79.2	4.61
			115	96	38.06	32.98	38.27	43.37	10.39	32-41	71.8	5.43
			93	81	37.02	31.51	37.31	41.97	10.46	34-43	74.2	5.09
	v 2	c 2	136	121	39.49	34.88	39.83	44.03	9.15	36-45	74.8	4.72
			86	73	38.75	33.50	39.56	43.94	10.44	32-41	69.8	4.88
		c 3	75	68	38.29	33.18	38.35	43.43	10.25	34-43	69.1	4.65
		c 4	85	71	39.26	34.07	39.39	44.45	10.38	36-45	69.1	5.19
			130	104	39.17	34.81	38.91	43.17	8.36	36-45	78.7	4.90
			94	80	38.75	34.27	38.50	43.15	8.88	34-43	77.6	4.98

TABLE 13 (continued)

SUMMARY OF DATA: SPEED PARAMETERS

LOCATION STATION	FACTOR	SIGN COND.	N	n	MEAN SPEED	SPEED PERCENTILES			SPEED DIFF.	10 MPH PACE		STD. DEV.
						V_{15}	V_{50}	V_{85}		RANGE	PER CENT	
A 14	t 1	c 1	93	75	37.19	32.31	35.89	42.95	10.64	32-41	69.3	5.59
		c 2	281	234	38.43	33.89	38.49	42.89	9.00	34-43	76.5	4.52
			213	175	38.07	32.97	38.31	42.86	9.79	32-41	72.6	4.66
		c 3	169	152	38.33	33.44	38.24	43.02	9.58	34-43	72.2	4.70
		c 4	178	148	39.05	34.32	38.74	43.93	9.61	34-43	73.6	4.89
			245	200	38.64	33.78	38.65	43.21	9.43	34-43	73.5	5.19
			187	161	37.88	33.32	37.93	42.36	9.04	34-43	75.9	5.11
A 16	v 1	c 4	167	116	33.42	29.40	33.26	37.28	7.88	28-37	81.1	3.90
		c 5	156	111	33.92	29.21	33.35	38.08	8.87	28-37	80.2	4.78
			170	115	32.78	28.10	32.36	37.31	9.21	28-37	73.9	4.96
			178	126	33.28	29.25	32.91	36.94	7.69	28-37	84.0	4.31
	v 2	c 4	145	106	33.88	30.08	33.88	37.50	7.42	30-39	84.1	4.05
		c 5	85	64	33.72	29.69	33.05	37.40	7.71	28-37	82.9	4.29
			89	63	33.68	28.74	34.00	37.79	9.05	28-37	73.0	4.64
			121	92	33.22	29.20	33.51	36.92	7.72	28-37	81.5	3.81

TABLE 13 (continued)

SUMMARY OF DATA: SPEED PARAMETERS

LOCATION	FACTOR	SIGN	N	n	MEAN	SPEED PERCENTILES			SPEED	10 MPH PACE		STD.
STATION		COND.			SPEED	V_{15}	V_{50}	V_{85}	DIFF.	RANGE	PER CENT	DEV.
A 16	t 1	c 1	119	80	35.70	29.51	35.74	42.36	11.85	30-39	55.0	6.47
		c 4	312	222	33.66	29.75	33.58	37.37	7.62	30-39	81.7	3.97
		c 5	241	175	33.85	29.42	33.24	37.82	8.40	28-37	81.1	4.61
			259	178	33.10	28.26	32.76	37.45	9.19	28-37	73.6	4.86
			299	218	33.25	29.22	33.16	36.92	7.70	28-37	83.0	4.11
A 17	v 1	c 1	138	106	33.50	28.56	33.61	37.85	9.29	28-37	77.5	4.40
		c 4	133	97	32.00	27.42	31.78	36.71	9.29	26-35	76.3	4.29
			138	97	32.30	28.41	32.37	36.09	7.68	26-35	59.4	3.79
			106	78	31.60	26.72	30.76	37.19	10.47	26-35	74.3	4.89
			135	93	32.78	28.68	32.77	36.88	8.20	30-39	80.8	4.02
	v 2	c 1	97	75	33.57	28.40	33.68	38.86	10.46	30-39	69.3	5.96
		c 4	97	82	32.79	28.09	32.78	37.10	9.01	28-37	76.9	4.35
			86	66	32.74	29.13	32.76	36.26	7.13	28-37	84.9	3.38
			103	80	31.00	26.09	30.37	36.50	10.41	26-35	71.2	5.31
			103	82	33.18	29.34	32.81	37.23	7.89	28-37	80.4	4.22

TABLE 13 (continued)

SUMMARY OF DATA: SPEED PARAMETERS

LOCATION STATION	FACTOR	SIGN COND.	N	n	MEAN SPEED	SPEED PERCENTILES			SPEED DIFF.	10 MPH PACE		STD. DEV.
						V_{15}	V_{50}	V_{85}		RANGE	PER CENT	
A 17	t 1	c 1	104	89	30.82	26.49	30.42	35.64	9.15	26-35	75.4	4.44
			235	181	33.53	27.60	33.64	38.39	10.79	28-37	73.0	4.64
		c 4	230	179	32.37	27.73	32.22	36.94	9.21	28-37	74.9	4.33
			224	163	32.48	28.78	32.55	36.23	7.45	28-37	80.9	3.67
			209	158	31.30	26.43	30.59	36.83	10.40	26-35	72.8	5.15
			238	175	32.97	29.07	32.79	37.06	7.99	28-37	78.2	4.12
B	v 1	65	30	23	46.24	36.97	47.02	52.53	15.56	44-53	61.2	7.82
		SZA	32	25	48.18	39.25	47.00	59.75	20.50	44-53	56.0	7.94
		50	38	32	45.88	40.03	45.75	51.63	11.60	44-53	71.9	6.56
		50	56	40	41.80	32.50	40.50	52.83	10.33	34-43	65.0	9.04
		40	50	39	35.32	31.76	35.60	39.56	7.80	32-41	84.8	3.82

TABLE 13 (continued)

SUMMARY OF DATA: SPEED PARAMETERS

LOCATION STATION	FACTOR	SIGN COND.	N	n	MEAN SPEED	SPEED PERCENTILES			SPEED DIFF.	10 MPH PACE		STD. DEV.
						V_{15}	V_{50}	V_{85}		RANGE	PER CENT	
B	v 2	65	56	46	50.11	43.85	49.82	58.57	14.72	44-53	58.7	7.10
		SZA	57	47	46.20	40.84	45.39	53.41	12.57	42-51	63.8	6.42
		50	64	52	47.42	41.29	47.26	54.24	12.95	42-51	57.6	6.22
		50	69	45	43.21	37.67	43.88	49.61	11.94	38-47	64.5	5.66
		40	110	68	34.53	30.14	34.12	39.44	9.30	30-39	75.0	4.34
	t 1	65	71	69	48.82	40.95	49.14	55.93	14.98	44-53	59.5	7.56
		SZA	89	72	46.89	40.22	46.17	53.99	13.77	40-49	54.1	7.08
		50	102	84	46.83	39.60	46.50	53.33	13.73	44-53	60.7	6.40
		50	125	85	42.55	34.76	42.11	50.61	15.85	38-47	50.7	7.48
		40	160	107	34.82	30.41	34.65	39.50	9.09	30-39	74.7	4.18

TABLE 13 (continued)

SUMMARY OF DATA: SPEED PARAMETERS

LOCATION STATION	FACTOR	SIGN COND.	N	n	MEAN SPEED	SPEED PERCENTILES			SPEED DIFF.	10 MPH PACE		STD. DEV.
						V_{15}	V_{50}	V_{85}		RANGE	PER CENT	
B	t 2	65	71	66	44.17	35.81	43.01	52.55	16.74	36–45	48.5	8.02
		SZA	80	70	47.61	40.48	47.70	54.50	14.02	46–55	54.2	7.16
		50	64	56	46.86	39.97	45.75	55.88	15.91	35–47	55.3	7.46
		50	135	100	39.42	31.50	39.13	45.83	14.33	34–43	58.6	7.08
		40	128	95	35.85	31.56	35.79	39.86	8.30	32–41	78.9	4.50
C	v 1	65	64	50	50.22	42.25	50.30	59.17	16.92	46–55	46.0	7.37
		40	75	59	39.45	33.78	39.20	45.68	11.90	34–43	66.0	5.62
		30	109	88	29.54	24.86	29.36	33.68	8.82	24–33	76.2	4.54
	v 2	65	73	44	54.64	45.21	56.49	62.47	17.26	54–63	47.7	8.11
		40	54	35	43.19	36.76	43.01	51.00	14.24	38–47	54.3	6.83
		30	86	66	30.44	25.20	30.51	34.95	9.75	26–35	71.3	4.98

TABLE 13 (continued)

SUMMARY OF DATA: SPEED PARAMETERS

LOCATION STATION	FACTOR	SIGN COND.	N	n	MEAN SPEED	SPEED PERCENTILES			SPEED DIFF.	10 MPH PACE		STD. DEV.
						V_{15}	V_{50}	V_{85}		RANGE	PER CENT	
C	t 1	65	137	94	52.69	43.50	52.25	60.79	17.29	50-59	41.6	8.04
		40	129	94	40.84	34.25	40.40	48.67	14.42	36-45	57.6	6.36
		30	195	154	29.93	25.00	29.78	34.24	9.24	24-33	74.2	4.75
	t 2	65	126	99	47.75	37.40	48.46	55.40	18.00	46-55	44.5	9.68
		40	137	101	42.66	35.24	42.42	50.45	15.21	36-45	55.4	6.72
		30	171	89	28.90	24.02	28.36	34.14	10.12	22-31	69.6	4.64
D	v 1	65	143	89	48.00	42.66	47.98	53.91	11.25	44-53	67.4	5.82
			90	68	47.82	40.86	47.48	55.73	14.87	44-53	58.6	6.37
		SZA	84	53	48.52	40.62	48.00	54.86	14.24	44-53	52.7	7.60
			104	82	46.99	41.28	46.11	53.37	12.09	42-51	63.5	6.27
		40	129	83	42.23	35.51	41.54	48.25	12.74	36-45	59.1	5.75
			98	76	44.10	36.77	43.16	50.52	13.75	36-45	55.3	6.86
		30	148	102	33.75	28.65	33.23	38.34	9.69	30-39	74.5	5.39
			154	121	34.81	29.35	34.61	40.05	10.70	30-39	67.0	4.88

TABLE 13 (continued)

SUMMARY OF DATA: SPEED PARAMETERS

LOCATION	FACTOR	SIGN	N	n	MEAN	SPEED PERCENTILES			SPEED	10 MPH PACE		STD.
STATION		COND.			SPEED	V_{15}	V_{50}	V_{85}	DIFF.	RANGE	PER CENT	DEV.
D	v 2	65	135	61	51.03	44.72	50.65	60.03	15.31	46-55	57.5	7.53
			109	63	53.10	45.98	53.01	60.52	14.54	52-61	54.0	6.19
		SZA	88	46	48.48	42.05	49.48	55.56	13.51	42-51	54.3	6.51
			92	61	47.35	41.08	46.52	54.60	13.52	42-51	55.6	6.59
		40	95	56	46.25	38.43	46.01	54.32	15.89	42-51	48.2	7.26
			103	68	44.03	36.57	44.38	50.79	14.22	42-51	52.9	6.50
		30	107	60	35.90	30.17	35.29	41.32	11.15	30-39	60.0	5.00
			88	62	35.60	30.57	35.30	40.39	9.82	30-39	74.3	4.90
	t 1	65	278	150	49.15	43.08	49.69	56.27	13.19	44-53	61.8	6.62
			199	131	50.36	43.81	50.03	57.69	13.88	44-53	54.2	6.81
		SZA	172	99	48.50	40.96	48.57	55.14	14.18	44-53	51.6	7.12
			196	143	47.14	41.20	46.24	54.10	12.90	42-51	60.1	6.41
		40	224	139	43.85	36.08	43.56	51.32	15.24	40-49	51.8	6.80
			201	144	44.07	36.69	43.75	50.69	14.00	42-51	52.7	6.68
		30	255	162	34.55	29.40	34.01	39.58	10.18	30-39	69.2	5.34
			242	183	35.08	29.86	34.83	40.17	10.31	30-39	69.5	4.89

TABLE 13 (continued)

SUMMARY OF DATA: SPEED PARAMETERS

LOCATION STATION	FACTOR	SIGN COND.	N	n	MEAN SPEED	SPEED PERCENTILES V_{15}	V_{50}	V_{85}	SPEED DIFF.	10 MPH PACE RANGE	PER CENT	STD. DEV.
D	t 2	65	207	129	42.78	35.36	42.97	49.27	13.91	38-47	58.1	6.31
		SZA	231	157	39.44	34.23	39.33	45.06	10.83	34-43	64.4	6.07
		40	242	147	39.48	34.24	39.46	44.64	10.40	34-43	69.4	5.05
		30	248	192	31.83	27.31	29.65	36.67	9.36	28-37	73.4	4.84
E	v 1	SZA	101	59	44.09	38.59	43.41	50.42	11.83	38-47	67.8	5.74
			124	64	42.03	36.31	41.19	47.84	11.53	38-47	65.5	6.46
		40	105	56	41.47	36.25	40.78	47.09	10.84	36-45	71.4	4.94
			126	90	40.57	33.38	39.49	46.76	13.38	36-45	58.9	6.20
		30	154	72	28.17	23.60	27.63	32.41	8.81	24-33	81.9	3.70
			129	75	28.77	24.54	28.10	33.05	8.51	24-33	82.7	4.16

TABLE 13 (continued)

SUMMARY OF DATA: SPEED PARAMETERS

LOCATION STATION	FACTOR	SIGN COND.	N	n	MEAN SPEED	SPEED PERCENTILES			SPEED DIFF.	10 MPH PACE		STD. DEV.
						V_{15}	V_{50}	V_{85}		RANGE	PER CENT	
E	v 2	SZA	110	57	47.45	39.84	47.07	54.99	15.15	42-51	54.2	7.37
			141	81	42.94	36.20	43.19	49.81	13.61	42-51	58.1	5.79
		40	130	73	42.61	38.15	41.88	48.23	10.08	38-47	72.5	4.87
			138	71	40.78	35.64	40.63	46.28	10.64	34-43	71.9	4.82
		30	127	59	28.33	24.27	28.69	31.57	7.30	24-33	84.7	3.49
			130	72	28.47	24.19	28.22	33.56	9.37	22-31	76.3	4.12
	t 1	SZA	211	116	45.74	39.02	45.10	52.04	13.02	38-47	56.9	6.80
			265	145	42.54	36.23	42.09	49.34	13.11	36-45	60.0	6.11
		40	235	129	42.11	37.22	41.39	47.72	10.50	36-45	70.5	4.92
			264	161	40.66	34.70	40.35	46.54	11.84	36-45	62.8	5.64
		30	281	131	28.24	24.36	28.20	31.05	6.69	24-33	83.2	3.62
			259	147	28.62	24.38	28.15	32.19	7.81	24-33	79.0	4.14
	t 2	SZA	155	119	44.48	35.60	44.50	51.55	15.95	42-51	49.6	7.30
		40	225	149	39.00	32.63	37.93	45.70	13.07	32-41	57.8	6.20
		30	273	161	29.84	25.65	29.35	34.66	9.01	26-35	76.5	4.33

TABLE 14

SUMMARY OF DATA: SPEED VIOLATIONS

Location Station	Factor	Sign Cond.	m	n	z	u
A 1	v 1	c 1	0	80	.00000	.0000
			0	70	.00000	.0000
			0	74	.00000	.0000
		c 7	0	60	.00000	.0000
	v 2	c 1	0	88	.00000	.0000
			0	90	.00000	.0000
			0	89	.00000	.0000
		c 7	0	95	.00000	.0000
	t 1	c 1	0	168	.00000	.0000
			0	160	.00000	.0000
			0	163	.00000	.0000
		c 7	0	155	.00000	.0000
	t 2	c 1	0	101	.00000	.0000
			0	143	.00000	.0000
		c 7	0	138	.00000	.0000
A 2	v 1	c 1	1	87	.01149	.2148
			0	60	.00000	.0000
			3	68	.04412	.4232
		c 6	5	77	.06494	.5152
		c 7	3	74	.04054	.4054
			1	59	.01695	.2612
			4	60	.06667	.5224
			11	72	.15278	.8030

TABLE 14 (continued)

SUMMARY OF DATA: SPEED VIOLATIONS

Location Station	Factor	Sign Cond.	m	n	z	u
A 2	v 2	c 1	1	93	.01075	.2078
			5	96	.05208	.4604
			14	101	.13861	.7632
		c 6	6	92	.06522	.5166
		c 7	8	107	.07477	.5538
			4	112	.03571	.3806
			8	95	.08421	.5888
			11	112	.09821	.6376
	t 1	c 1	2	180	.01111	.2112
			5	186	.03205	.3600
			17	169	.10059	.6456
		c 6	11	169	.06509	.5160
		c 7	11	181	.06077	.4982
			5	171	.02924	.3436
			12	145	.08276	.5836
			22	184	.11957	.7060
	t 2	c 1	14	195	.07179	.5424
			4	165	.02424	.3126
			9	208	.04327	.4190
		c 6	5	147	.03401	.3910
		c 7	3	138	.02174	.2960
			4	126	.03175	.3584
			4	97	.04124	.4090
			3	102	.02941	.3446

TABLE 14 (continued)

SUMMARY OF DATA: SPEED VIOLATIONS

Location Station	Factor	Sign Cond.	m	n	z	u
A 3	v 1	c 1	3	87	.03448	.3736
			2	110	.01818	.2704
		c 2	1	101	.00990	.1994
			1	85	.01176	.2172
			8	100	.08000	.5734
			3	80	.03750	.3896
			2	86	.02326	.3062
			5	103	.04854	.4442
		c 6	2	88	.02273	.3026
		c 7	1	78	.01282	.2268
			1	115	.00870	.1870
			0	87	.00000	.0000
			5	63	.07937	.5712
			4	84	.04762	.4400
	v 2	c 1	3	64	.04688	.4374
			6	83	.07229	.5340
		c 2	4	84	.04762	.4400
			3	104	.02885	.3414
			2	82	.02439	.3134
			7	104	.06731	.5260
			10	102	.09804	.6370
			1	97	.01031	.2034
		c 6	3	98	.03061	.3518
		c 7	4	95	.04211	.4134
			2	98	.02041	.2866
			3	103	.02913	.3432
			13	98	.13265	.7456
			6	81	.07407	.5512

TABLE 14 (continued)

SUMMARY OF DATA: SPEED VIOLATIONS

Location Station	Factor	Sign Cond.	m	n	z	u
A 3	t 1	c 1	6	151	.03974	.4014
			8	193	.04145	.4154
		c 2	5	185	.02703	.3306
			4	189	.02116	.2922
			10	182	.05495	.4732
			10	184	.05435	.4706
			12	188	.06383	.5108
			6	200	.03000	.3482
		c 6	5	186	.02688	.3292
		c 7	5	173	.02890	.3416
			3	213	.01408	.2380
			3	190	.01579	.2518
			18	161	.11180	.6820
			10	165	.06060	.4976
	t 2	c 1	14	124	.11290	.6854
			8	203	.03941	.3996
		c 2	5	168	.02762	.3340
			9	146	.06164	.5018
			9	133	.06767	.5262
			5	169	.02959	.3460
			8	146	.05479	.4724
			5	123	.04065	.4060
		c 6	7	195	.03590	.3814
		c 7	7	191	.03665	.3852
			5	146	.03425	.3724
			8	145	.05517	.4740
			7	126	.05556	.4758
			12	101	.11881	.7036

TABLE 14 (continued)

SUMMARY OF DATA: SPEED VIOLATIONS

Location Station	Factor	Sign Cond.	m	n	z	u
A 4	v 1	c 1	4	116	.03448	.3734
		c 2	4	107	.03738	.3890
		c 3	3	79	.03797	.3922
			3	92	.03261	.3632
			1	110	.00909	.1910
		c 7	2	109	.01835	.2718
	v 2	c 1	8	90	.08889	.6054
		c 2	5	76	.06579	.5188
		c 3	4	85	.04706	.4372
			12	110	.10909	.6728
			4	103	.03883	.3966
		c 7	9	117	.07692	.5620
	t 1	c 1	12	206	.05825	.4874
		c 2	9	183	.04918	.4472
		c 3	7	164	.04268	.4162
			15	202	.07426	.5520
			5	213	.02347	.3074
		c 7	11	226	.04867	.4448

TABLE 14 (continued)

SUMMARY OF DATA: SPEED VIOLATIONS

Location Station	Factor	Sign Cond.	m	n	z	u
A 4	t 2	c 1	9	146	.06164	.5018
		c 2	13	169	.07692	.5620
		c 3	9	187	.04813	.4424
			8	128	.06202	.5032
		c 7	9	158	.05696	.4820
A 5	v 1	c 1	2	103	.01942	.2796
		c 2	2	147	.01361	.2340
		c 3	4	137	.02920	.3434
			2	104	.01923	.2782
		c 7	3	115	.02609	.3244
	v 2	c 1	3	79	.03797	.3922
		c 2	7	93	.07527	.5558
		c 3	11	118	.09322	.6208
			2	82	.02439	.3134
		c 7	3	85	.03529	.3778
	t 1	c 1	5	182	.02747	.3330
		c 2	9	240	.03750	.3896
		c 3	15	255	.05882	.4898
			4	186	.02151	.2944
		c 7	6	200	.03000	.3482

TABLE 14 (continued)

SUMMARY OF DATA: SPEED VIOLATIONS

Location Station	Factor	Sign Cond.	m	n	z	u
A 5	t 2	c 1	4	195	.02051	.2876
		c 2	6	206	.02913	.3428
		c 3	6	195	.03077	.3526
		c 7	4	182	.02198	.2976
A 6	v 1	c 1	2	143	.01399	.2372
		c 3	4	129	.03101	.3542
		c 5	1	126	.00794	.1784
		c 6	1	143	.00699	.1674
		c 7	1 3 7	118 116 114	.00847 .02586 .06140	.1844 .3230 .5008
	v 2	c 1	4	116	.03448	.3736
		c 3	10	121	.08264	.5830
		c 5	6	93	.06452	.5136
		c 6	2	115	.01739	.2646
		c 7	3 9 4	92 100 103	.03261 .09000 .03883	.3632 .6094 .3966

TABLE 14 (continued)

SUMMARY OF DATA: SPEED VIOLATIONS

Location Station	Factor	Sign Cond.	m	n	z	u
A 6	t 1	c 1	6	259	.02317	.3056
		c 3	14	250	.05600	.4778
		c 5	7	219	.03196	.3596
		c 6	3	258	.01163	.2160
		c 7	4	210	.01905	.2768
			12	216	.05556	.4758
			11	217	.05069	.4540
	t 2	c 1	4	186	.02151	.2944
		c 3	5	175	.02857	.3396
		c 5	3	217	.01382	.2358
		c 6	2	211	.00948	.1954
		c 7	10	147	.06803	.5280
			8	119	.06723	.5246
			6	141	.04255	.4156
A 7	v 1	c 1	3	155	.01935	.2790
		c 2	7	145	.04828	.4430
		c 3	5	142	.03521	.3774
		c 4	6	144	.04167	.4106
			2	119	.01681	.2602
		c 7	3	133	.02256	.3016

TABLE 14 (continued)

SUMMARY OF DATA: SPEED VIOLATIONS

Location Station	Factor	Sign Cond.	m	n	z	u
A 7	v 2	c 1	3	80	.03750	.3896
		c 2	1	91	.01099	.2100
		c 3	1	100	.01000	.2004
		c 4	5	90	.05556	.4758
			5	99	.05051	.4532
		c 7	1	129	.00775	.1764
	t 1	c 1	6	235	.02553	.3208
		c 2	8	236	.03390	.3704
		c 3	6	242	.02479	.3162
		c 4	11	234	.04701	.4370
			7	218	.03211	.3604
		c 7	4	262	.01527	.2478
	t 2	c 1	9	201	.04478	.4264
		c 2	5	220	.02273	.3026
		c 3	5	220	.02273	.3026
		c 4	8	201	.03980	.4016
		c 7	15	170	.08824	.6030

TABLE 14 (continued)

SUMMARY OF DATA: SPEED VIOLATIONS

Location Station	Factor	Sign Cond.	m	n	z	u
A 11	t 1	c 1	5	85	.05892	.4902
		c 5	8	111	.07207	.5434
		c 6	5	113	.04425	.4240
			5	141	.03546	.3788
A 14	t 1	c 1	7	75	.09333	.6210
		c 2	11	234	.04701	.4370
			8	175	.04571	.4310
		c 3	11	152	.07237	.5448
		c 4	12	148	.08108	.5774
			18	200	.09000	.6094
			8	161	.04969	.4496
A 16	t 1	c 1	15	80	.18750	.8956
		c 4	7	222	.03153	.3570
		c 5	13	175	.07429	.5520
			7	178	.03933	.3992
			8	218	.03670	.3856
A 17	t 1	c 1	14	89	.15730	.8156
			56	181	.30939	1.1796
		c 4	42	179	.23464	1.0114
			29	163	.17791	.8714
			29	158	.18354	.8854
			41	175	.23429	1.0104

TABLE 14 (continued

SUMMARY OF DATA: SPEED VIOLATIONS

Location Station	Factor	Sign Cond.	m	n	z	u
B	v 1	65	0	23	.00000	.0000
		SZA	0	25	.00000	.0000
		50	2	32	.06250	.5054
		50	3	40	.07500	.5550
		40	0	39	.00000	.0000
	v 2	65	0	46	.00000	.0000
		SZA	0	47	.00000	.0000
		50	4	52	.07692	.5620
		50	1	45	.02222	.2994
		40	0	68	.00000	.0000
	t 1	65	0	69	.00000	.0000
		SZA	0	72	.00000	.0000
		50	6	84	.07143	.5412
		50	4	85	.04706	.4376
		40	0	107	.00000	.0000

TABLE 14 (continued)

SUMMARY OF DATA: SPEED VIOLATIONS

Location Station	Factor	Sign Cond.	m	n	z	u
B	t 2	65	0	66	.00000	.0000
		SZA	0	70	.00000	.0000
		50	9	56	.16071	.8248
		50	2	100	.02000	.2838
		40	1	95	.01053	.2056
C	v 1	65	0	50	.00000	.0000
		40	9	59	.15254	.8024
		30	6	88	.06818	.5284
	v 2	65	2	44	.04545	.4296
		40	13	35	.37143	1.3108
		30	8	66	.12121	.7112
	t 1	65	2	94	.02128	.2928
		40	22	94	.23404	1.0098
		30	14	154	.09091	.6126
	t 2	65	2	99	.02020	.2816
		40	29	101	.28713	1.1312
		30	10	89	.11236	.6838

TABLE 14 (continued)

SUMMARY OF DATA: SPEED VIOLATIONS

Location Station	Factor	Sign Cond.	m	n	z	u
D	v 1	65	0	89	.00000	.0000
			0	68	.00000	.0000
		SZA	0	53	.00000	.0000
			0	82	.00000	.0000
		40	22	83	.26506	1.0818
			29	76	.38158	1.3318
		30	35	102	.34314	1.2516
			52	121	.42975	1.4298
	v 2	65	1	61	.01639	.2568
			0	63	.00000	.0000
		SZA	0	46	.00000	.0000
			0	61	.00000	.0000
		40	29	56	.51786	1.6064
			29	68	.42647	1.4234
		30	29	60	.48333	1.5374
			30	62	.48387	1.5386
	t 1	65	1	150	.00667	.1636
			0	131	.00000	.0000
		SZA	0	99	.00000	.0000
			0	143	.00000	.0000
		40	51	139	.36691	1.3016
			58	144	.40278	1.3772
		30	64	162	.39506	1.3596
			82	183	.44809	1.4670

TABLE 14 (continued)

SUMMARY OF DATA: SPEED VIOLATIONS

Location Station	Factor	Sign Cond.	m	n	z	u
D	t 2	65	0	129	.00000	.0000
		SZA	0	157	.00000	.0000
		40	16	147	.10884	.6724
		30	40	192	.20833	.9480
E	v 1	SZA	0	59	.00000	.0000
			0	64	.00000	.0000
		40	11	56	.19643	.9182
			18	90	.20000	.9272
		30	2	72	.02778	.3348
			8	75	.10667	.6634
	v 2	SZA	1	57	.01754	.2658
			0	81	.00000	.0000
		40	18	73	.24658	1.0394
			13	71	.18310	.8844
		30	1	59	.01695	.2646
			4	72	.05556	.4760
	t 1	SZA	1	116	.00862	.1860
			0	145	.00000	.0000
		40	29	129	.22481	.9880
			31	161	.19255	.9086
		30	3	131	.02290	.3038
			12	147	.08163	.5796

TABLE 14 (continued)

SUMMARY OF DATA: SPEED VIOLATIONS

Location Station	Factor	Sign Cond.	m	n	z	u
E	t 2	SZA	0	119	.00000	.0000
		40	23	149	.15436	.8078
		30	16	161	.09938	.6416

APPENDIX C

ILLUSTRATIVE EXAMPLE

In order to understand more easily the special method used in the statistical analysis, an illustrative example with actual speed data is carried through. It is regarded superfluous to deal with the treatment of the frequency distributions to obtain the speed parameters desired. Any good textbook in applied statistics accomplishes that purpose, for example Burr (79) or Duncan (81). Powers (84) gives also a good description on analysis of frequency distributions related specifically to speed data.

Calculation of Error Mean Square

The illustrative example chosen deals with the significance test of the 85th percentile speed at station A 4. Table 15 shows the 85th percentile speeds recorded for various sign conditions by vehicle factor. This is a case with three replications in one cell and one in the others. The test method utilized requires one measurement per cell; consequently, the three measurements have to be converted into one by obtaining a weighted average. The procedure is outlined in Table 16. By definition the weighted mean is

$$\overline{X} = \frac{\sum_{i=1}^{n} n_i X_i}{\sum_{i=1}^{n} n_i}$$

TABLE 15

85TH PERCENTILE SPEED AT STATION A 4

	c 1	c 2	c 3	c 7
v 1	39.42	40.77	41.80 41.32 40.68	41.58
v 2	43.01	43.67	42.90 43.98 41.43	42.99

TABLE 16

CALCULATION OF WEIGHTED MEAN AND SUM OF SQUARES FOR REPLICATED DATA

v 1			
n_i	X_i	n_iX_i	$n_iX_i^2$
79	41.80	3,302.20	138,031.9600
92	41.32	3,801.44	157,075.5008
110	40.68	4,474.80	182,034.8640
281		11,578.44	477,142.3248

TABLE 16 (continued)

CALCULATION OF WEIGHTED MEAN AND SUM OF SQUARES FOR REPLICATED DATA

v 2			
n_i	X_i	n_iX_i	$n_iX_i^2$
85	42.90	3,646.50	156,434.8500
110	43.98	4,837.80	212,766.4440
103	41.43	4,267.29	176,793.8247
298		12,751.59	545,995.1187

For v 1: $\bar{X} = \frac{11,578.44}{281} = 41.2044$

For v 2: $\bar{X} = \frac{12,751.59}{298} = 42.7905$

From the same table the within cell sum of squares, SSg, is obtained, defined by (see also page 53)

$$SSg = \sum n_i X_i^2 - \bar{X} \sum n_i X_i$$

For v 1: $SSg = 477,142.3248 - (41.2044)(11,578.44) = 59.6517$

For v 2: $SSg = 545,995.1187 - (42.7905)(12,751.59) = 348.2068$

The total sum of squares within all the cells, SSk, is defined by

$$SSk = \sum SSg$$

Thus $SSk = 59.6517 + 348.2068 = 407.8585$

Dividing SSk with the sum of the degrees of freedom for each cell, respectively, an estimate is obtained of the mean square, MSk, based on

individual measurements

$$MSk = \frac{SSk}{\sum (g - 1)}$$

In this example $MSk = \frac{407.8585}{2 + 2} = 101.9646$

The error mean square, MSe, which, in fact, is a correction of MSk because of the reduction of replications within the cells is defined by

$$MSe = \frac{MSk}{\bar{n}}$$

where $\bar{n}$ is the harmonic mean of all the n_i's within the k cells:

$$\bar{n} = \frac{1}{\frac{1}{k} \sum_{k} \left[\frac{1}{\sum_{i=1}^{g} n_i} \right]}$$

The procedure of obtaining the harmonic mean is outlined in Table 17.

TABLE 17

CALCULATION OF HARMONIC MEAN

Sign Condition	v 1		v 2	
	n	1/n	n	1/n
c 1	116	.008621	90	.011111
c 2	107	.009346	76	.013158
c 3	281	.003559	298	.003356
c 7	109	.009174	117	.008547

$$\Sigma(1/n) = .066872$$

$$\tilde{n} = \frac{1}{1/8 \cdot (.066872)} = 119.6315$$

Using that value of $\overline{n}$

$$MSe = \frac{101.9646}{119.6315} = .8523$$

However, this value is correct only if the variable X_i from Table 16 represents the mean speed. For speed percentiles a correction factor has to be applied, the value of which is given in Table 10. For the 85th percentile speed this factor is 2.3501. Accordingly, the error mean square for the 85th percentile speed is

$$MSe = \frac{0.8523}{2.3501} = .3627$$

This method, generally termed "unweighted mean's method," is thoroughly outlined by Snedecor (86).

Analysis of Variance

The second part of the analysis procedure is the analysis of variance, in this case, a two-way factorial design with one replication per cell. Table 18 delineates the general procedure of obtaining the various sum of squares.

The hypothesis we wish to test is that there is no difference between the speeds neither by condition nor by vehicle factor. Consequently, the alternative hypothesis is that there is a difference between the speeds. This significance test is delineated in a so-called

TABLE 18

CALCULATION OF SUM OF SQUARES IN ANOVA

	c 1	c 2	c 3	c 7	$\sum$
v 1	39.42	40.77	41.20	41.58	162.97
v 2	43.01	43.67	42.79	42.99	172.46
$\sum$	82.43	84.44	83.99	84.57	335.43

$$(T)^2 = (335.43)^2 = 112{,}513.2849$$

$$(Tc)^2 = (82.43)^2 + \ldots + (84.57)^2 = 28{,}131.2235$$

$$(Tv)^2 = (162.97)^2 + (172.46)^2 = 56{,}301.6725$$

$$(Tcv)^2 = (39.42)^2 + \ldots + (42.99)^2 = 14{,}078.5189$$

$$SSc = \frac{(Tc)^2}{2} - \frac{(T)^2}{8} = 14{,}065.6117 - 14{,}064.1606 = 1.4511$$

$$SSv = \frac{(Tv)^2}{4} - \frac{(T)^2}{8} = 14{,}075.4181 - 14{,}064.1606 = 11.2575$$

$$SScv = (Tcv)^2 - \frac{(T)^2}{8} - SSc - SSv =$$

$$= 14{,}078.5189 - 14{,}064.1606 - 1.4511 - 11.2575 = 1.6497$$

ANOVA table, Table 19. The result of the test is readily distinguished; the hypothesis that there is no significant difference between local and non-local vehicle speeds is rejected on a five per cent significance level due to the fact that the calculated F-value is greater, and in

TABLE 19

SIGNIFICANCE TEST

Source of Variation	df	Sum of Squares	Mean Square	F	$F_{.05}$
Condition c	3	1.4511	0.4837	1.3336	6.5914
Vehicle v	1	11.2575	11.2575	31.0380	7.7086
Interaction cxv	3	1.6497	0.5499	1.5161	6.5914
Error	4		0.3627		

this case even substantially greater, than the theoretical F-value. On the other hand, the hypothesis is accepted as to sign conditions and interaction.

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LOCATING SLIPPERY HIGHWAY SITES BY ACCIDENT ANALYSIS

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JUNE 1960
NO. 10

by

V.G. STOVER
&
H.L. MICHAEL

Joint
Highway
Research
Project

PURDUE UNIVERSITY
LAFAYETTE INDIANA

Technical Paper

LOCATING SLIPPERY HIGHWAY SITES
BY ACCIDENT ANALYSIS

TO:	K. B. Woods, Director Joint Highway Research Project	June 8, 1960
FROM:	H. L. Michael, Assistant Director Joint Highway Research Project	File: 8-5-5

A technical paper entitled, "Locating Slippery Highway Sites by Accident Analysis" by V. G. Stover and H. L. Michael is attached. This paper was presented to the 46th Annual Purdue Road School in April 1960 at the research general session.

The paper is a summary of the material contained in the final report entitled, "The Location of Slippery Highway Sites by Accident Analysis" which was previously presented to the Board. It reports an economical and efficient method for locating slippery highway surfaces and for determining their priority for "dealicking". The method appears to be potentially important in Indiana and in other states in the attack on the slippery highway surface problem.

The paper is presented for the record and for release for publication in the Proceedings of the Road School and possibly in other technical publications.

Respectfully submitted,

Harold L Michael

Harold L. Michael, Secretary

HLM:kmc

Attachment

cc: F. L. Ashbaucher
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F. F. Havey
G. A. Leonards
J. F. McLaughlin
R. D. Miles
R. E. Mills
C. E. Vogelgesang
J. L. Waling
J. E. Wilson
E. J. Yoder

Technical Paper

Locating Slippery Highway Sites by Accident Analysis

by

V. G. Stover, Graduate Assistant
and
H. L. Michael, Assistant Director

Joint Highway Research Project
File: 8-5-5
Project: C-36-59E

Purdue University
Lafayette, Indiana

June 8, 1960

LOCATING SLIPPERY HIGHWAY SITES BY ACCIDENT ANALYSIS

INTRODUCTION

It is generally agreed that a wet pavement surface presents less resistance to skidding than does the same surface in a dry condition. It is also well known that ice and snow are very slippery, but on the other hand they are special conditions which do not reflect any skid characteristic of the pavement surface. It is, therefore, the wet surface condition that is critical relative to the minimum coefficient of friction that will be provided by a particular pavement surface.

In many instances a surface which does not indicate any particular difficulty with respect to skidding when dry will become quite slippery when wet and accidents as a consequence of slippery surfaces may result. The accident spot map of Indiana for 1958 is shown in Figure 1. Figure 2 shows the dry surface accidents occurring in a portion of Grant County during 1958. It is rather typical of the location of accidents occurring on dry surfaces anywhere in the state. There is a concentration of accidents on SR 9 south and west of Marion; however, there is no particular grouping of skidding accidents.

Figure 3 shows the same area of Grant County but indicates the wet surface accidents occurring in 1958. Here again there is a concentration of accidents on the heavily traveled portion of SR 9. However, a rather definite grouping of skidding accidents is now obvious at the junction of SR 9 and SR 15 as well as at the junction of SR 9 and SR 37. Although skidding was involved in several dry surface accidents throughout the shown portion of the county there was no indication that any sites were

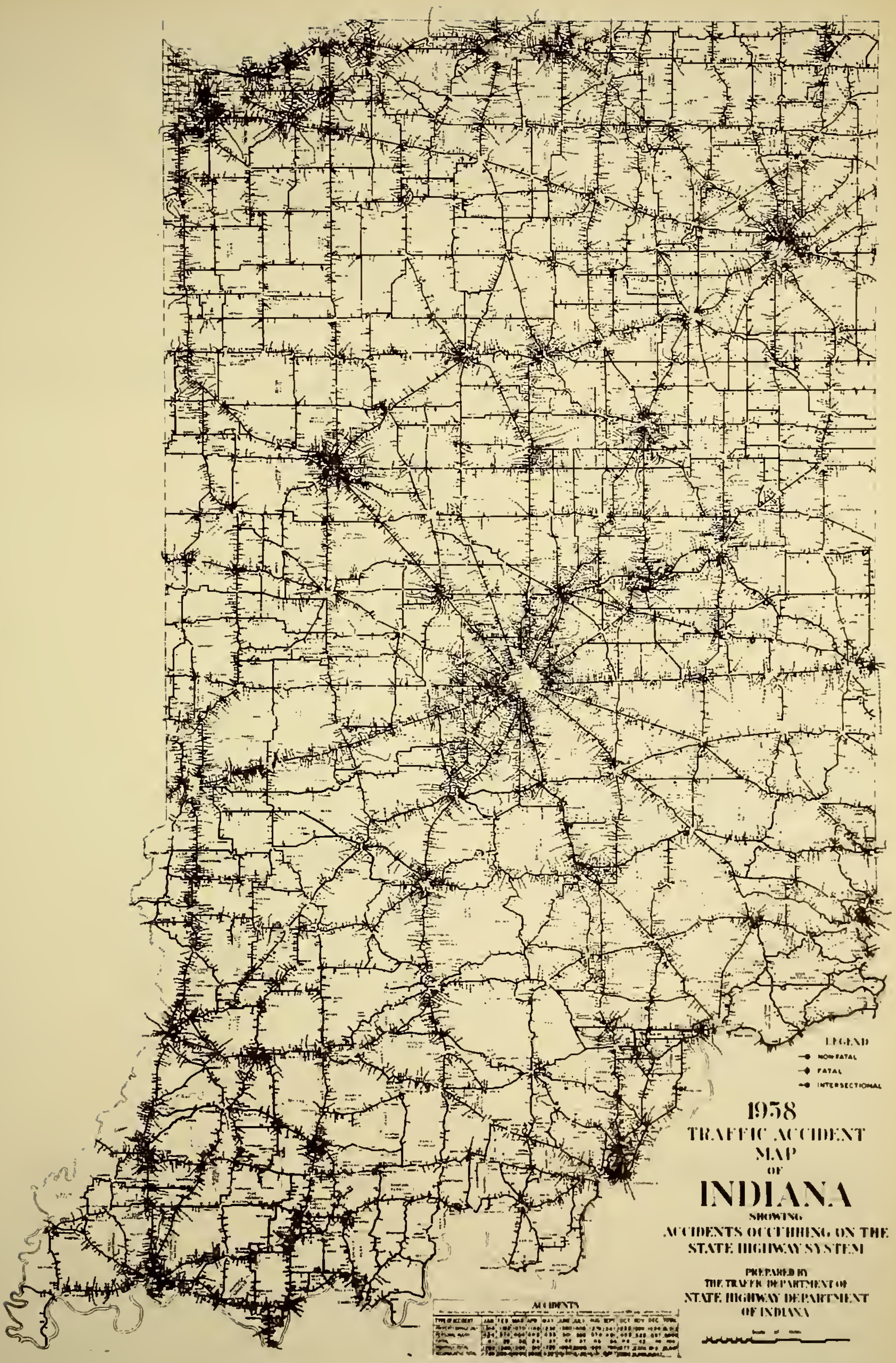
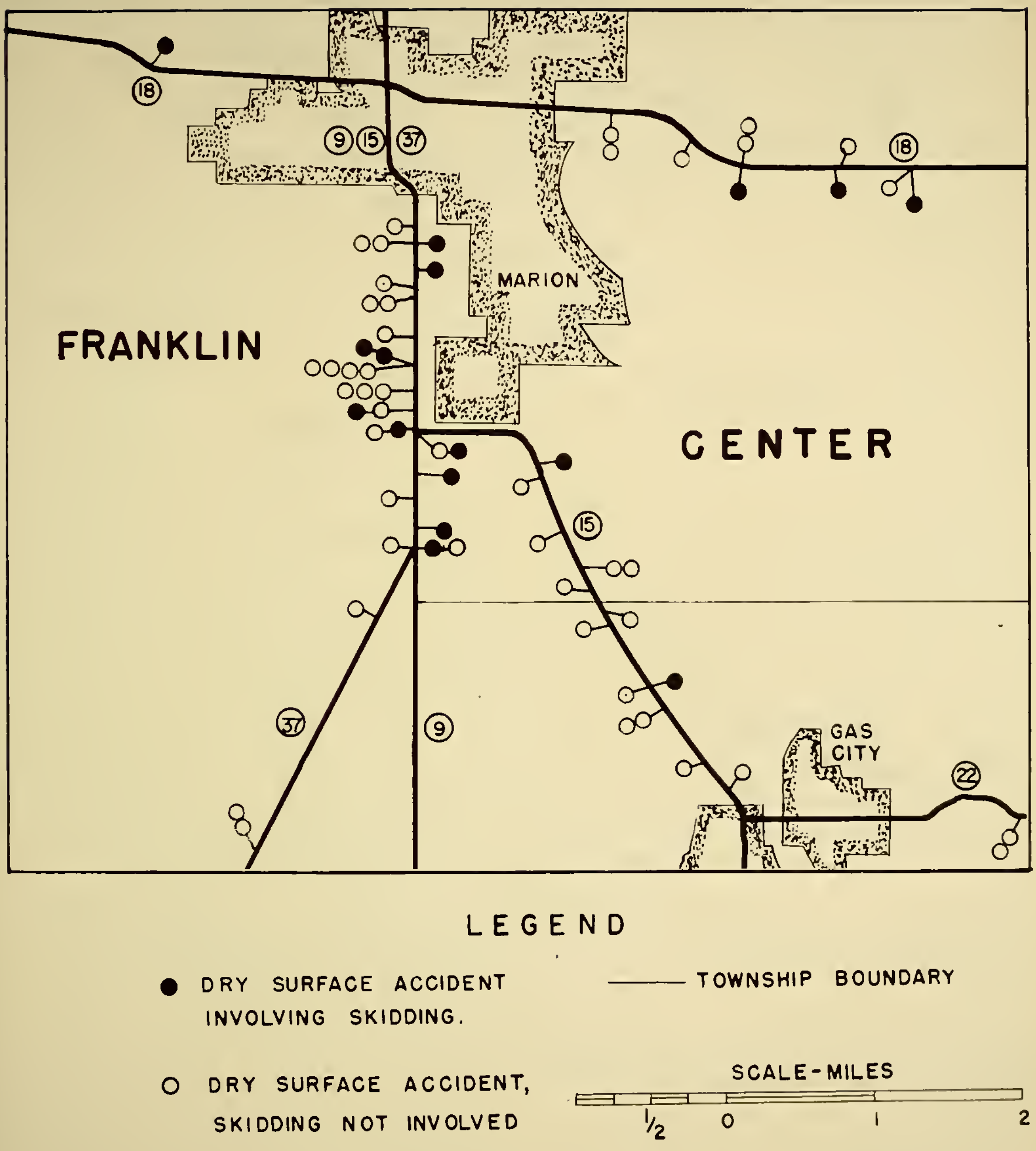
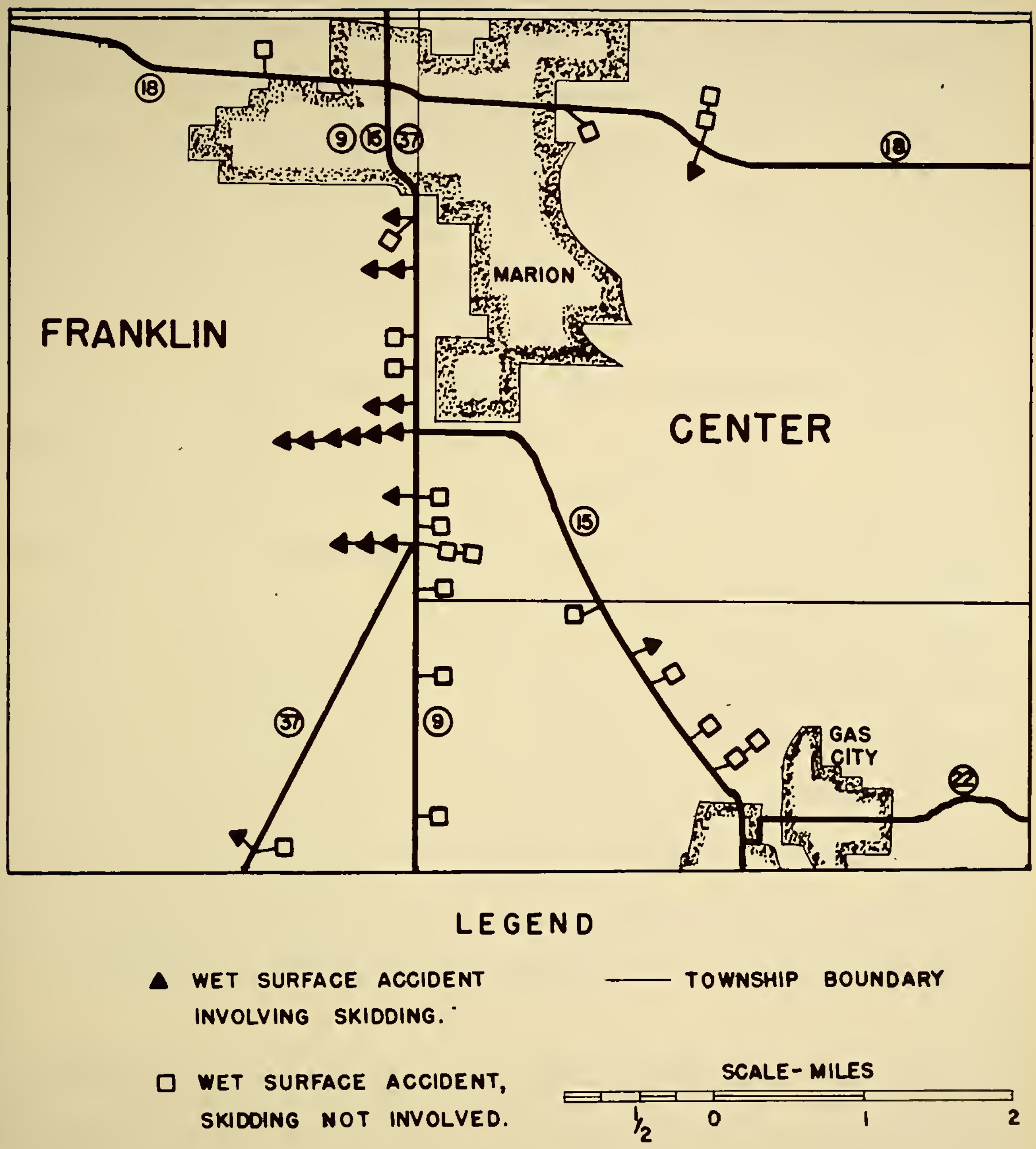

LEGEND
NON-FATAL
FATAL
INTERSECTIONAL
1958
TRAFFIC ACCIDENT
MAP
OF
INDIANA
SHOWING
ACCIDENTS OCCURRING ON THE
STATE HIGHWAY SYSTEM
PREPARED BY
THE TRAFFIC DEPARTMENT OF
STATE HIGHWAY DEPARTMENT
OF INDIANA
ACCIDENTS

FIGURE 1


FRANKLIN
CENTER
MARION
GAS CITY
18
9
15
37
22
LEGEND
● DRY SURFACE ACCIDENT INVOLVING SKIDDING.
○ DRY SURFACE ACCIDENT, SKIDDING NOT INVOLVED
TOWNSHIP BOUNDARY
SCALE-MILES
½
0
1
2

PORTION OF GRANT COUNTY, INDIANA

FIGURE 2


18
9
15
37
MARION
18
FRANKLIN
CENTER
15
37
9
GAS CITY
22
LEGEND
▲ WET SURFACE ACCIDENT INVOLVING SKIDDING.
□ WET SURFACE ACCIDENT, SKIDDING NOT INVOLVED.
TOWNSHIP BOUNDARY
SCALE- MILES
½
0
1
2

PORTION OF GRANT COUNTY, INDIANA

FIGURE 3

experiencing difficulty with respect to skidding. In Figure 3, however, it is noted that the wet surface accidents which involved skidding are clustered at those sites at which the drivers may have been required to make several difficult maneuvers and where the wet surface presented a reduced resistance to skidding that was below that necessary to complete the maneuvers safely. Skid tests conducted on SR 9 just north of its junction with SR 15 indicated an average distance to stop of 98 feet from 30 mph; a vehicle traveling at the average speed for this section of highway (40 mph) would require about 200 feet to stop.

It is not intended to convey the impression here that the road is the only part of the driver-vehicle-road system that contributes to skidding. It is, however, the only component over which the highway engineer has direct and substantial control. Moreover, by applying sound engineering practices to the highway it may be possible to minimize the deficiencies of the driver and the vehicle. The efficient and effective use of non-slippery highway surfaces for the purpose of reducing the number of accidents involving skidding requires that slippery sites at which accidents are occurring be identified and evaluated at the earliest possible date.

The commonly-employed field testing procedures (such as the stopping-distance method) effectively evaluate on a relative basis the skid resistance of the surfaces upon which the tests are conducted. They are, however, expensive and time consuming. Furthermore, some other method must be utilized to determine the locations that are suspected of being slippery and at which the field tests should be conducted. A procedure which would systematicly evaluate the skid resistance of segments of the

entire highway system and which would indicate those sites which are "slippery" would be of considerable value. In the interest of economy it would be desirable if such a method would minimize the number of field tests required and would necessitate a minimum of labor.

The accident report files contain a wealth of information and it was this source that this study used. Previous investigations conducted in Great Britain (1, 2)* developed and utilized statistical methods for the analysis of highway sites having a low frequency of skidding accidents. After-studies conducted at sites that had been so located and then "de-slicked" indicated a substantial reduction in skidding accidents as well as in total accidents (3).

SKIDDING IN REPORTED ACCIDENTS

The over-all problem of skidding in accidents occurring in 1958, as determined from the reported accidents on the state highways in ten counties of Indiana, is summarized in Table 1.

In order to compare the number of accidents by surface condition it is necessary to consider the amount of traffic which traversed these highways while the surfaces were dry, wet, or "icy" (i.e., covered with ice or snow). The number of vehicle-miles traveled under the different surface conditions is unknown and is impossible to obtain. However, there must be a relationship between the total number of accidents occurring while the surfaces were dry, or "icy" and the proportion of the time the surfaces were in each of these conditions as well as the amount of traffic using these facilities while these conditions prevailed. Therefore, if

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* Numbers in parentheses refer to listings in the bibliography.

TABLE 1

SKIDDING AS A FACTOR IN REPORTED ACCIDENTS

1958 Data, Ten Counties in Indiana

Condition of Road Surface	Total Number of Accidents	Number of Accidents in which Skidding was Involved	Skidding* Rate
Dry	1776	713	41
Wet	779	469	60
Icy (i.e., covered with ice or snow)	275	199	72
Total	2830	1381	49

* The number of accidents involving skidding as a percentage of the total number of accidents.

the total number of accidents occurring while the surface was dry, wet, or "icy" is used as a measure of the exposure, a measure of the relative risk of skidding can be obtained by comparing the skidding rate (i.e., the percentage of accidents involving skidding).

The rather high skidding rate on dry surfaces is not surprising since most vehicles have braking systems of sufficient capacity to lock the wheels (4). Such locking is especially true when the driver perceives an accident as being imminent and attempts a "panic stop" and/or when the brakes are improperly adjusted. The skidding rate on wet surfaces, however, is considerably higher than on dry surfaces and it is still higher when the surface is "icy".

EVALUATION OF LOW FREQUENCY ACCIDENT SITES

The skidding rate is a useful method of indicating the relative risk of skidding on dry, wet, and "icy" surfaces because a large number of accidents have occurred. The accident frequency of the vast majority of specific sites is quite small, however, and some other method is necessary to consider the effect of chance occurrence and to determine when the frequency of skidding accidents is excessive. Such a method must also enable the highway engineer to determine when the frequency of skidding accidents is greater than some acceptable standard on some other length of highway. Statistical methods enable the solution of problems of this nature.

The statistical analysis used in this study is based on the binomial distribution which is a statistical distribution of occurrences which exhibit a particular characteristic which might be classified as either a success or a failure. With respect to skidding accidents this characteristic is skidding; an accident which did not involve skidding can be classified as a success and an accident in which skidding was involved can be classified as a failure. The minimum percentage of accidents in which skidding was, in all probability, involved can be calculated; this value will be referred to as the significant skidding rate or the SSR. In other words the long term value of the skidding rate is, in all probability, at least as large as the calculated SSR.

As an example of the necessity for utilizing this procedure for comparing low accident frequencies let us consider two sites, both of which have the same skidding rate. Such a situation is illustrated in Figure 4; "n" is the total number of accidents which occurred when the surface was wet, and "X" is the number of these accidents in which skidding occurred.

SITE "A"

n = 2

X = 2

SKIDDING RATE = 100%

SIGNIFICANT SKIDDING RATE = 15%

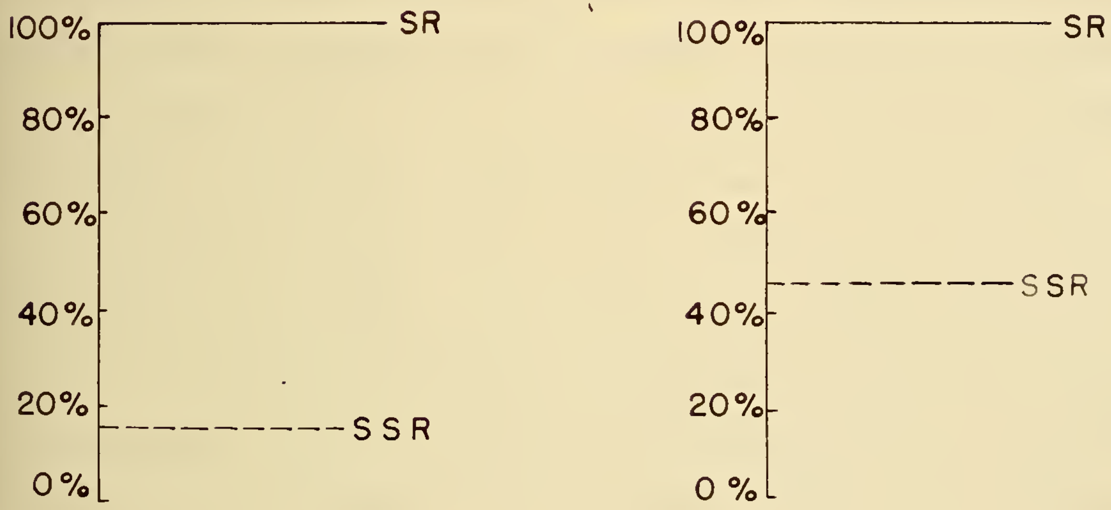
SITE "B"

n = 4

X = 4

SKIDDING RATE = 100%

SIGNIFICANT SKIDDING RATE = 45%



THE SIGNIFICANCE OF LOW ACCIDENT FREQUENCIES

GRAPHICALLY REPRESENTED

FIGURE 4

In the case of site "A" two out of two wet surface accidents involved skidding; therefore, the skidding rate is 100 percent. However, the two out of two may be due to chance, and we can only say, at the 5 percent significant level, that 95 percent of the time (19 times out of 20) we would expect the true long term skidding rate at this location to be at least 15 percent. It might be as high as 100 percent, but the chances are small (one chance in 20) that it is less than 15 percent. In the case of site "B" four out of four wet surface accidents involved skidding; thus, the skidding rate for this site is also 100 percent. However, since the frequency is higher (i.e., there are more accidents involved) we can determine the minimum value that the long-term skidding rate will have within narrower limits (i.e., the effect of chance occurrence is less). The SSR for this site is 45 percent. Here again the true skidding rate might be as high as 100 percent but the chances are only one in 20 that it is less than 45 percent. Thus, we see that the SSR is dependent upon the accident frequency and is in reality the lower confidence limit for the skidding rate.

As an aid in determining the SSR Table 2 has been prepared for a wide range of accident frequencies. Tables of the Cumulative Binomial Probability Distribution (5) were used to calculate the values indicated in the table and are for the five percent significance level. The table is most easily used by entering the table with the number of wet-surface accidents involving skidding (column X), then reading horizontally to the column farthest to the right which has a number larger than the total number of wet surface accidents which occurred. The SSR is then given at the top of this column. For example, if a total of seven wet surface accidents occurred and four

TABLE 2

TABLE FOR THE DETERMINATION OF THE SIGNIFICANT SKIDDING RATE

Number of Wet Surface Accidents Which Skidding Occurred X	The Skidding Rate* is significantly greater than the percentage given at the top of the column if the total number of wet surface accidents is less than that given in the body of the table.															Number of Wet Surface Accidents in Which Skidding Occurred X
	2%	4%	6%	8%	10%	12%	14%	15%	20%	25%	30%	35%	40%	45%	50%	
2	19	10	7	5	4	4	3	3	——	——	——	——	——	——	——	2
3	42	22	15	11	9	8	7	7	5	4	4	4	——	——	——	3
4	69	35	24	18	15	13	11	10	8	7	6	5	5	5	——	4
5	100	51	34	26	21	18	16	15	11	10	8	7	7	6	6	5
6	132	67	45	34	28	24	20	19	15	12	11	9	9	8	7	6
7	166	84	57	43	35	29	25	24	18	15	13	11	10	10	9	7
8	201	101	68	52	42	35	31	29	22	18	15	14	12	11	10	8
9	236	119	80	61	49	41	36	34	26	21	18	16	14	13	12	9
10	273	138	93	70	57	48	41	39	30	24	21	18	16	15	14	10
11	310	157	105	80	64	54	47	44	33	27	23	20	18	17	15	11
12	348	175	118	89	72	60	52	49	37	30	26	23	20	18	17	12
13	387	194	131	99	80	67	57	53	41	34	29	25	22	20	19	13
14	426	214	144	109	87	73	63	59	45	37	31	27	24	22	20	14
15	465	234	157	119	95	80	69	65	49	40	34	30	26	24	22	15

TABLE 2 CONTINUED

X	2%	4%	6%	8%	10%	12%	14%	15%	20%	25%	30%	35%	40%	45%	50%	X
16	504	252	170	128	103	87	75	70	53	43	37	32	29	26	24	16
17	543	272	183	138	112	93	81	75	57	47	40	34	31	28	25	17
18	582	293	197	148	120	100	86	81	61	50	42	37	33	30	27	18
19	623	313	209	159	128	106	92	86	65	53	45	39	35	31	29	19
20	664	334	224	169	135	113	98	92	70	57	48	42	37	33	31	20
21	706	354	237	179	144	121	103	97	74	60	51	44	39	35	32	21
22	748	375	251	190	152	125	110	102	78	63	54	47	41	37	34	22
23	788	396	265	200	161	134	115	108	82	67	56	49	43	39	36	23
24	829	417	279	209	169	142	122	113	87	70	59	51	46	41	38	24
25	871	438	292	221	117	148	128	120	91	73	62	54	48	43	39	25
26		459	307	230	186	157	134	125	95	77	65	56	50	45	41	26
27		480	321	242	194	163	140	131	99	81	68	59	52	47	43	27
28		501	335	251	202	170	146	136	103	84	71	61	54	49	45	28
29		519	349	263	210	177	152	142	107	87	74	64	57	51	46	29
30		542	364	273	220	184	158	148	111	91	77	66	59	53	48	30
31			378	284	227	191	164	154	116	94	80	69	61	55	50	31
32			392	295	236	198	171	160	121	98	82	71	63	57	51	32
33			406	305	245	204	177	165	125	101	85	74	65	59	54	33

TABLE 2 CONTINUED

X	2%	4%	6%	8%	10%	12%	14%	15%	20%	25%	30%	35%	40%	45%	50%	X
34			421	316	253	211	183	171	130	104	88	76	68	61	55	34
35			435	327	263	220	189	177	133	107	91	79	70	63	57	35
36				338	270	226	195	183	138	111	94	81	72	65	59	36
37				349	281	233	202	188	142	114	97	84	74	67	61	37
38				361	288	242	206	194	147	118	100	87	77	69	63	38
39				370	297	248	212	201	151	122	102	89	79	71	64	39
40				382	306	255	221	205	155	125	105	92	81	73	66	40
41					315	263	225	210	161	129	108	94	83	75	68	41
42					324	270	231	217	164	132	111	97	85	77	70	42
43					332	278	240	223	169	136	114	99	88	79	72	43
44					342	285	245	228	173	140	117	101	90	81	73	44
45										143	121	103	92	83	75	45
46										147	123	106	95	85	77	46
47										151	126	109	97	87	79	47
48										154	130	112	99	89	81	48
49										157	132	114	101	91	83	49
50										162	135	116	102	93	84	50

* At the 0.05 level of significance (i.e., there is only one chance in twenty that the true skidding rate is less than that indicated).

of these involved skidding, one enters column X with four and reads horizontally to the last column containing a number larger than seven, which in this case is eight; the number at the top of this column gives the SSR as being 20 percent. This means that 19 times out of 20 the true long term percentage of wet surface accidents at this location which involve skidding is at least 20 percent.

APPLICATION OF THE METHOD

This method of analysis was applied to all the State and US marked routes in ten counties of Indiana; county roads were not included because many of them have gravel surfaces and were therefore outside the scope of the study. The SSR was determined for each accident cluster (i.e., a site where numerous accidents occurred) and for each road length, a road length being a numbered highway route throughout one township.

Table 3 shows the results of the road length analysis and indicates that a few of the roads are causing the bulk of the skidding problem. Similar results were obtained for the accident cluster analysis.

TABLE 3

DISTRIBUTION OF ROAD LENGTHS ACCORDING TO THE SSR

Significant Skidding Rate	Number of Road Lengths
50	9
40	7
30	16
20	14
10	46
0	175
Total =	267

In order to check the effectiveness and the reliability of the SSR as a measure of the slipperiness of the surface, field skid tests were conducted on a sample number of locations. These sample locations were selected by statistical means which insured that the results obtained truly represented all of the road lengths (6).

The skid tests were conducted with the State Highway Department of Indiana skid vehicle (7). This vehicle has an electrically-activated, vacum-braking unit and thus eliminates driver variation in the application of the brakes. It is also equipped so as to record the speed at which the vehicle was traveling when the brakes were applied and the distance required to stop. All tests were conducted on a thoroughly wet surface from 30 mph.

Upon the completion of the field skid testing the data were plotted as scatter diagrams. Visual inspection and some knowledge of the traffic conditions at the various locations indicated that speed and volume might be two additional factors which affect the SSR. Therefore, the over-all speeds were obtained for the sample locations by the license plate method (8). Although the speed data were obtained under dry rather than wet pavement conditions it is felt that little, if any, appreciable error is involved. Previous investigations (9) have found that even though speeds do drop when a rain-fall first begins, the drivers soon return to their former speed, provided that the intensity of rain-fall is not sufficient to cause a reduction in visability. The traffic volumes were obtained from the 1958 traffic flow map and the speed and volume factors were evaluated. The resulting scatter diagrams of the relationship between the SSR and the stopping distance, average speed and daily volume are

shown in Figures 5, 6 and 7, respectively, for the accident cluster analysis. Similar charts were obtained for the road length analysis.

The results of a correlation analysis for the accident cluster data is summarized in Table 4.

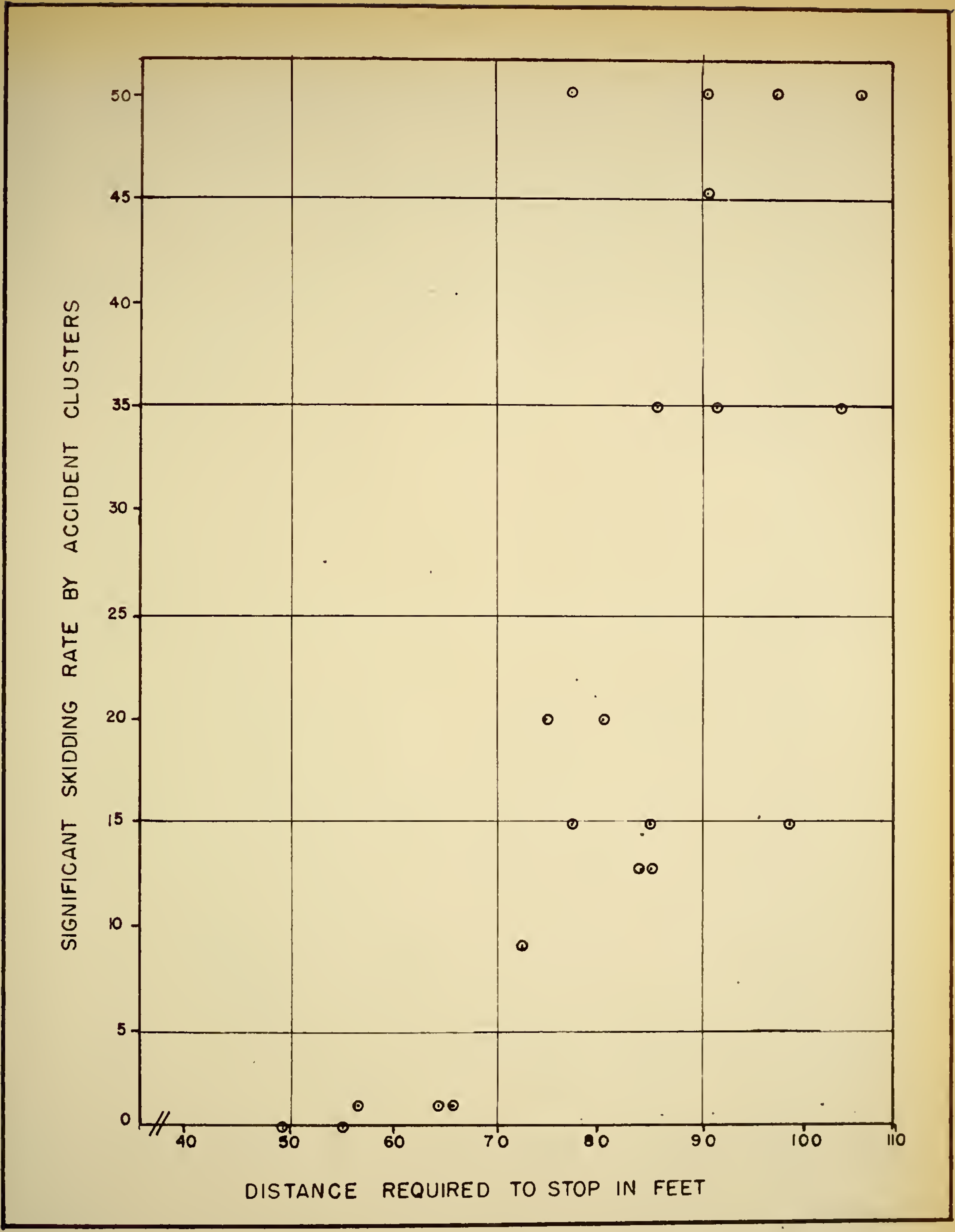
TABLE 4

SUMMARY OF THE CORRELATION ANALYSIS
FOR THE ACCIDENT CLUSTER DATA

Factors Correlated	Correlation Coefficient	Significant at the 0.05 Level
SSR - Slipperiness of the surface	$r_{y1} = 0.76$	yes
SSR - mean over-all speed	$r_{y2} = 0.27$	no
SSR - ADT	$r_{y3} = 0.58$	yes
SSR - Slipperniss of the surface, mean over-all speed, and ADT	$R = 0.86$	yes

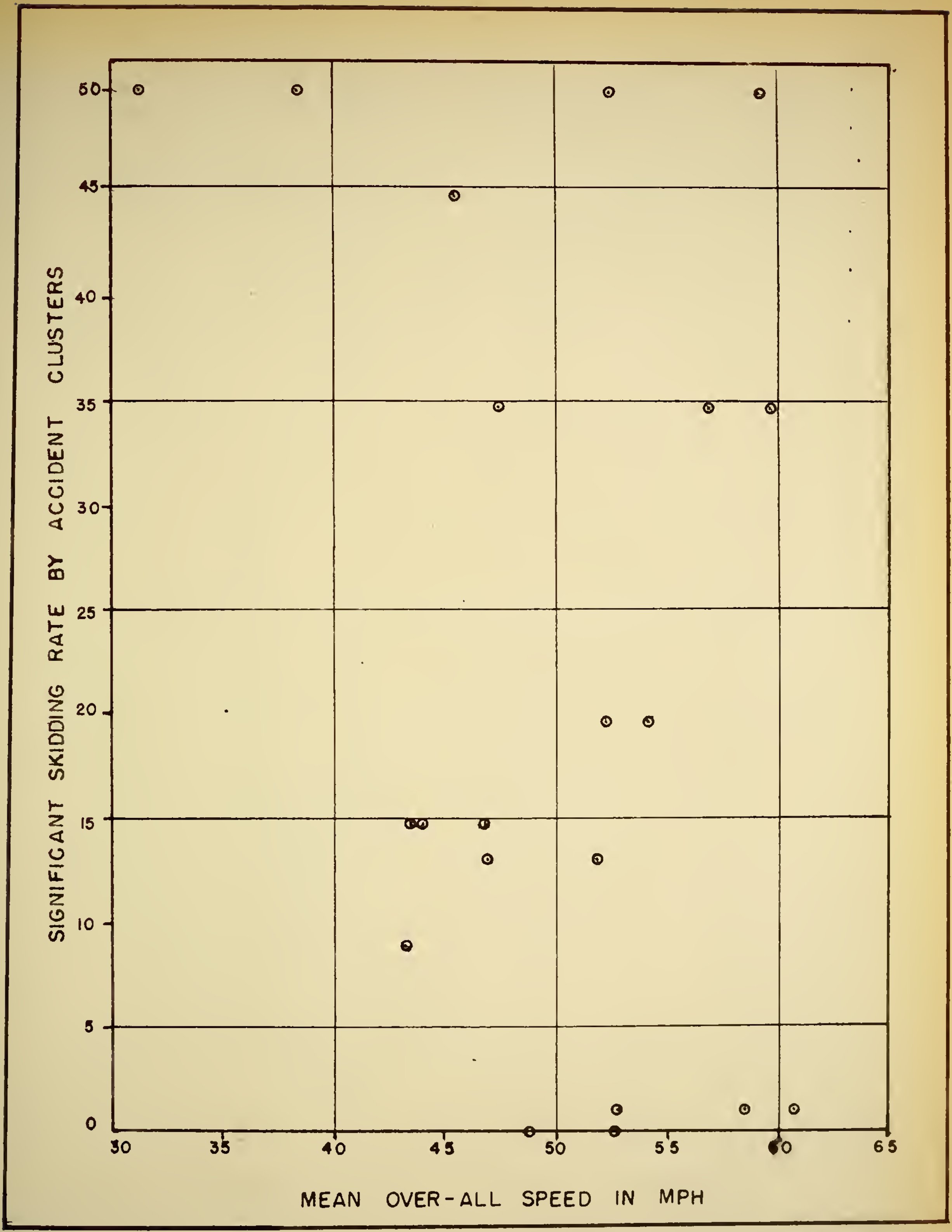
Although the correlation between the SSR and the mean over-all speed is not significant, visual inspection of the center portion of Figure 6 indicates a relationship between these two factors at some speeds. Further analysis indicated significant positive correlation between these two factors for the values of the SSR between five and 35.

In the examination of the simple correlation coefficients (the r_{yi} values) it must be remembered that they are a measure of the dependence between the two factors under consideration and may include influence of other variables. The multiple correlation coefficient (R) is perhaps more



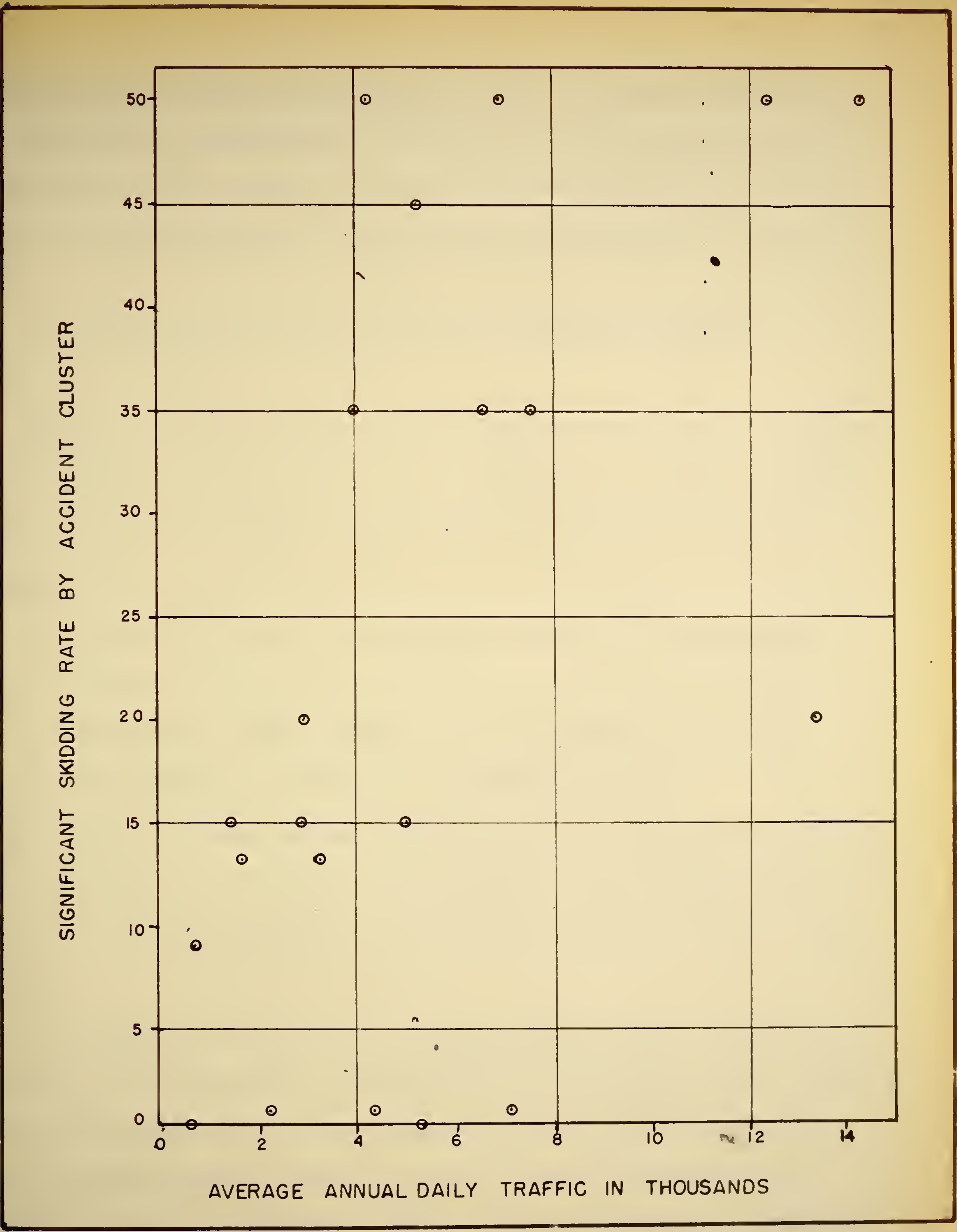
SCATTER DIAGRAM OF THE DISTANCE REQUIRED TO STOP AND THE SIGNIFICANT SKIDDING RATE BY ACCIDENT CLUSTER

FIGURE 5



SCATTER DIAGRAM OF THE MEAN OVER-ALL SPEEDS AND THE SIGNIFICANT SKIDDING RATE BY ACCIDENT CLUSTER

FIGURE 6



SCATTER DIAGRAM OF THE AVERAGE ANNUAL DAILY TRAFFIC AND THE SIGNIFICANT SKIDDING RATE BY ACCIDENT CLUSTER

FIGURE 7

valuable since it measures the degree of linear association between all the factors under consideration. The square of this coefficient is the portion of the total variation in the SSR (74 percent) that is dependent upon the distance required to stop, the mean over-all speed, and the ADT.

POSSIBLE PROCEDURES FOR A STATE-WIDE ROUTE SKID ANALYSIS

The question which now arises is: How might this procedure be utilized for the analysis of the paved streets and highways of a state? The first step necessary for locating slippery sites is to obtain the following information:

1. The total number of accidents which occurred when the pavement was wet.
2. The number of these accidents in which skidding was involved.
3. The locations at which these accidents occurred.

No differentiation should be made between skidding before and after braking; that is, the number of accidents in which skidding was involved should be the total number of accidents in which skidding, before and after braking, occurred.

The accident spot map is the best method for recording skidding accident data. With appropriate identification this information could be plotted on the accident map currently prepared in Indiana and in many other states. Visual inspection would then reveal those locations at which skidding accidents are grouping. A state-wide analysis could be performed as often as desired and individual troublesome locations could be analyzed as they become apparent.

As noted in Figure 1 this map is often some-what crowded. Therefore, it may be advantageous to maintain a separate map for wet surface accidents. It would then be a very simple matter to differentiate between those accidents which involved skidding and those which did not. This would necessitate a minimum of information on any one map and would, therefore, simplify initial visual inspection. The additional time required to plot the information necessary for a route skid-analysis would be quite nominal.

A separate location map for wet surface accidents might also be adapted so as to show the previous twelve months accidents rather than just those occurring since January 1st of any particular year. For example, this might be done by using pins with the numbers "1" through "12" on their heads to indicate the month in which the accident occurred. With such a system, the map could be studied to determine if any cluster existed at the end of each month and the pins representing the accidents which occurred during the earliest month of the preceeding year would then be removed. Such a procedure would permit an analysis with a full twelve months accident history at the end of each month and at any time for a minimum of eleven months.

For most efficient results, accurate information as to the location of the accident is required and care must be exercised when plotting the accident. For example, since the analysis should indicate which of the approaches to an intersection is slippery, the location of each accident must be accurately reported and plotted so as to indicate the proper approach on which the accident occurred instead of simply indicating that an accident occurred at a given intersection. Similarly, indication should be made as to the roadway on which each accident occurred on a multi-lane divided highway since one roadway may be "slick" while the other is not.

The greater the accuracy in reporting and plotting of the accidents the more accurately a cluster can be determined and, therefore, the shorter the length of highway that needs to be "de-slicked". Figure 3 shows the wet surface accidents which occurred in a portion of Grant County. Although a considerable length of SR 9 is probably slippery, the "de-slicking" of the approaches to the intersections of SR 9 with SR 15 and SR 9 with SR 37 could substantially reduced the problem of skidding as well as the total number of accidents in this area.

When the resistance to skidding is increased to a previously slippery site, this fact should be clearly noted. Accidents occurring after the corrective measure has been completed would continue to be plotted and the effectiveness of the "de-slicking" should be evaluated. The same procedure should be followed when any portion of a highway has had its surface characteristics changed through resurfacing in reconstruction.

The most beneficial results for the smallest expenditure of funds can be realized by the proper "de-slicking" of rather limited lengths of highway which have been identified as being slippery and troublesome by this method of accident report analysis.

RESULTS, CONCLUSIONS AND RECOMMENDATIONS

Several interesting and valuable results were obtained from the investigation; the more important are as follows:

1. There is an extremely good correlation between the SSR and the slipperiness of the surface as measured by the stopping-distance method. Therefore, the SSR is a reliable measure of the relative slipperiness of the wet highway surfaces.

2. There is a significant correlation between the SSR and ADT. The importance of this relationship is that the SSR inherently involves a consideration of the traffic volume. Therefore, separate study of the traffic volumes on the various facilities is not necessary when determining the priority that a particular section should be given for the application of a "de-slicking" treatment. Priority should be given to those sites which have the highest significant skidding rate.
3. Regardless of speed, skidding occurs in an alarmingly high percentage of wet surface accidents when a wet surface provides a low resistance to skidding. Conversely, a very low percentage of wet surface accidents will involve skidding if a wet surface exhibits a uniformly high resistance to skidding. In the region between these two limiting conditions there is a relationship between speed and skidding.
4. The use of the accident cluster is superior to using the road length. Better correlation between the SSR and the various factors affecting skidding can be expected when accident clusters are used. Furthermore, an analysis using the accident cluster will facilitate greater economy because shorter and more specific sites will be indicated as being in need of "de-slicking" treatment.
5. A location map of wet surface accidents is recommended as the most practical, flexible, and simplest method of identifying wet surface accident clusters and for obtaining the information necessary for performing an analysis utilizing this method.

The more accurately accidents are reported and plotted the more detailed the information obtained and, therefore, the greater the economy of the "de-slicking" operation.

6. It is recommended that slippery locations on the state highways of Indiana be determined at least once a year by the method just discussed and that high priority be given to proper "de-slicking" of these locations.

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PHOTOGRAMMETRIC MEASUREMENT OF FINAL PAY QUANTITIES IN HIGHWAY CONSTRUCTION

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R. R. JOHNSON
&
R. D. MILES

Joint
Highway
Research
Project

PURDUE UNIVERSITY
LAFAYETTE INDIANA

Technical Paper

PHOTOGRAMMETRIC MEASUREMENT OF FINAL PAY QUANTITIES IN HIGHWAY CONSTRUCTION

TO: K. B. Woods, Director
Joint Highway Research Project

June 8, 1960

FROM: H. L. Michael, Assistant Director
Joint Highway Research Project

File: 1-4-15

Attached is a technical paper entitled, "Photogrammetric Measurements of Final Pay Quantities in Highway Construction". This paper has been prepared by Mr. R. R. Johnson and Prof. R. D. Miles and was presented at the 1960 Purdue Road School.

It is submitted to the Board for the record and to request permission for its publication in the Proceedings of the Road School.

Respectfully submitted,

Harold L Michael

Harold L. Michael
Secretary

HLM:kmc

Attachment

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Technical Paper

PHOTOGRAMMETRIC MEASUREMENT OF FINAL PAY QUANTITIES IN HIGHWAY CONSTRUCTION

by

Russel R. Johnson
and
Robert D. Miles

Joint Highway Research Project
Project No. C-36-32 O
File No. 1-4-15

Purdue University
Lafayette, Indiana

June 1960

PHOTOGRAMMETRIC MEASUREMENT OF FINAL PAY QUANTITIES IN HIGHWAY CONSTRUCTION

INTRODUCTION

Photogrammetry is presently being used in the location and design of highways to achieve a saving in time, manpower, and cost. Considerable interest has been expressed in extending the use of photogrammetry to achieve a similar saving in the measurement of highway final pay quantities. The acceptance of photogrammetric methods for the determination of final pay quantities, however, is contingent upon their applicability in this respect and upon the accuracy that can be obtained for each type of measurement.

This paper reports the results of an attempt to apply photogrammetric methods to the determination of several final pay quantities and to evaluate the accuracy obtained by comparison with quantities determined by normal field procedures. The final pay quantities concerned were earthwork, concrete pavement and appurtenances, paved side ditch, sodding, curbing, guard rail, and guide posts. The study was performed on a newly constructed section of highway approximately 10,000 feet in length. The photogrammetric plotting was accomplished with a standard six-inch focal length Kelsh plotter at a scale of 1 inch = 50 feet.

Photogrammetric spot elevation readings for the final roadbed cross sections and field survey notes furnished by the State Highway Department for the original terrain cross sections were used in combination to determine the earthwork quantities. Comparisons were analyzed on the basis of volumes for individual sections, for individual plotter models, for various classifications of excavation and embankment, for urban and

rural classifications, and for the entire project. Although payment is made on the basis of excavation quantities only, embankment quantities were also considered as it was felt that they would aid in the evaluation of earthwork quantities in general. Computation of the earthwork quantities was accomplished with an IBM 650 electronic computer.

Since vertical accuracy is of great importance in the determination of earthwork quantities, a statistical analysis of vertical accuracy was undertaken by comparing photogrammetric centerline elevations with the corresponding elevations from the field survey.

Non-earthwork final pay items were delineated on the photogrammetric manuscript, and the quantities were determined by scaling or planimetering. These quantities were compared with the corresponding quantities determined by normal field procedures.

Present Methods

The present practice in Indiana regarding final earthwork quantities is to make payment on the basis of cubic yards of excavation as measured in the original position by taking cross sections before excavation is started and again after it is completed. Volumes are computed by the average end-area method. If the cost of excavation is specifically included in the payment for any item of work, the final cross sections are taken at the finished surface of the work. Payment for embankment is not ordinarily made on a unit volume basis, but is included in the various pay items of the contract such as spreading and compacting of embankment material, labor and equipment.

Other final pay quantities such as pavement, curbing, paved side ditch, guard rail, and sodding are measured when complete in-place and

accepted, with payment being made on a contract unit price per lineal foot or square yard.

Photogrammetric Methods

The proposed procedure for determining earthwork quantities photogrammetrically is a direct analogue of the field survey method. Cross section lines are drawn on the photogrammetric manuscript at right angles to the centerline. Spot elevations along the cross section lines are read directly from the photogrammetric plotter or the elevations may be interpolated from photogrammetrically established contours on the manuscript. For final pay measurements, it is generally agreed that spot elevations would be more desirable since they can be made with at least twice the precision of the interpolated readings. The coordinates of each reading are written on the manuscript at the position of the reading.

The ease and facility of reading elevations from the plotter, as opposed to the laborious and time consuming field methods, motivates the measurement of more cross sections and more elevations per cross section. This is highly desirable since the accuracy of the terrain representation is a function of the density of elevation readings.

In connection with the measurement of non-earthwork quantities, it may be said that the physical dimensions of any object identifiable to its full extent in the plotter model can be measured. The measurement of these quantities, however, is not always in the horizontal and vertical system of coordinates: a condition which calls for special allowances when making photogrammetric measurements since direct measurements from the plotter are always horizontal and vertical.

Study Location

Several factors were considered in the selection of a section of highway on which to perform the investigation. The study site had to be

recently completed construction and of sufficient length to provide an adequate statistical base for accuracy evaluation. The topography, land use, and proximity to Lafayette were deemed desirable considerations, but not necessarily of a controlling nature.

A newly constructed section of State Road 27 beginning at Liberty, Indiana and extending northward about five miles was selected as a suitable location for the study. A portion of the study area showing the overall terrain is illustrated in Figure 1.

The construction on State Road 27 was primarily of a relocation nature although some of the highway was rebuilt to higher standards in the same position as before construction. Construction was completed in the fall of 1958. The actual study was performed on a 9,643 foot length of this section beginning at Westcott Street in the town of Liberty, Indina and proceeding northward. About 800 feet of the study section may be assigned an urban land use classification as it is within the city limits of Liberty.

The terrain may be described as rolling although there are some flat reaches. Some of the excavations and embankments were relatively large as would be expected in terrain of this nature.

Quantities Measured

It was decided early in the study that the quantities which could possibly be measured were earthwork, concrete pavement, concrete pavement appurtenances, paved side ditch, sodding, guard rail, guide post, and curbing. Only items which could be identified and delineated from air-photos were considered. Payment for such quantities had to be on the basis of dimension measure rather than weight measure. On this premise, a quantity

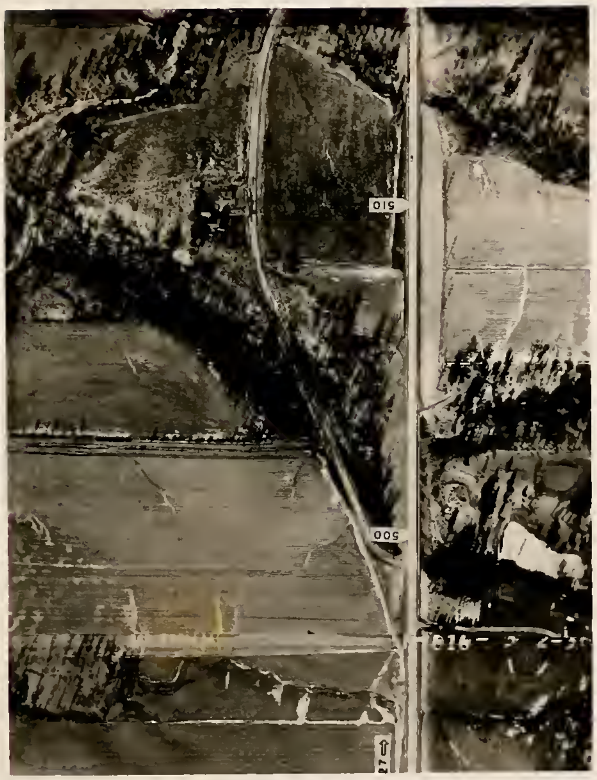


Figure 1. A section of recently constructed highway used in the investigation

measurement such as seeding would not qualify for photogrammetric measurement since payment for this item is on the basis of weight measurement of seed, fertilizer and straw mulching. Nor would base course materials lend themselves to photogrammetric measurement since these quantities are not visible from air-photos.

Procedures

The photography was flown at an altitude of 1,500 feet above average ground elevation by the State Highway Department. This altitude provided a contact scale of 1 inch = 250 feet and a manuscript plotting scale of 1 inch = 50 feet. The diapositives were made on .06-inch sensitized glass plates printed emulsion side up. The photography and diapositives were, in general, of high quality. The corners of some photos were slightly underexposed, but as most cross section plotting was near the center of the photos, only the observation of elevation control points in the darker corners was adversely affected.

The planimetric outline of the pavement was first drawn on the manuscript by plotting points about one inch apart to delineate the centerline and edges. These points were then connected by using a straight edge or highway curves on the curved portions.

Cross section lines were drawn at right angles to the centerline at the proper centerline stations as indicated by the highway level books. Correct positioning of the cross section stations was achieved by referencing permanent station marks, which were stamped on the pavement at 500 foot intervals, to the horizontal ground control.

Spot elevations were read to 0.1 foot along the cross section lines at the centerline station, pavement edges, and at significant changes in

the profile of the cross section line. The elevation readings were extended far enough on either side of the centerline to include any terrain which indicated earthwork movements. The position of the reading point was marked on the cross section line, and the elevation and the scaled distance of the point from the centerline were recorded on the right and left of the point respectively as shown in Figure 2.

Special borrow pits were plotted in the same manner, except that the cross section lines were referenced to a base line rather than the centerline of the highway.

The planimetric features of other final pay items were delineated on the manuscript by guiding the tracing table so that the floating mark and plotting pencil followed the feature being compiled. The dimensions of these items were not recorded on the manuscript at the time of plotting but were later measured and tabulated upon the completion of each model. Large irregular-shaped areas, such as sodding, were planimetered several times and averaged to increase the accuracy of measurement. Areas of more constant dimensions were subdivided into simple geometric shapes for scaling and computation. Paved side ditch, guard rail and curbing were measured in the linear direction only. Guide posts were outlined and counted.

Cross sectional areas for earthwork quantities were determined by a combination of photogrammetric and conventional methods. The original cross sections of the terrain before construction were procured by field survey methods; whereas, the template sections (or, cross sections of the finished road bed) were procured by photogrammetric spot elevation readings. Field level books from the highway department supplied the data for the original cross sections. Although it would have been possible, and perhaps

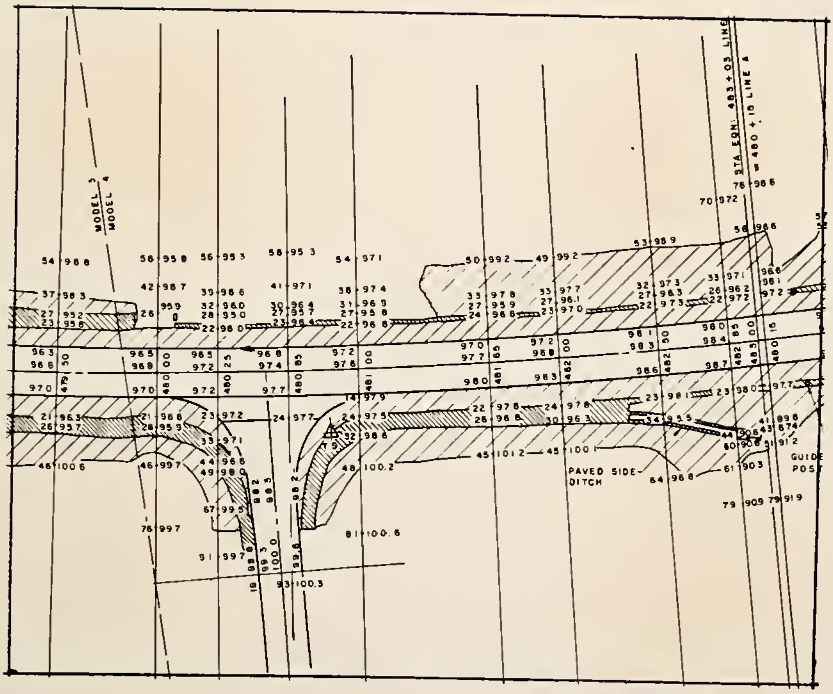


Figure 2. Section of photogrammetric manuscript.

desirable, to determine the initial terrain data by photogrammetric methods, the time interval from staking of the centerline to completion of construction would have precluded such a procedure for this study.

Studies conducted in California (2,3) indicated that the accuracy of photogrammetric earthwork quantities could be considerably improved by adjusting cross section elevation readings to an accurate centerline profile. It was decided that this procedure should be included in the study along with the method of determining earthwork quantities without adjustments. Accordingly, all template sections were raised or lowered by an amount equal to the error at the centerline station, and the resulting earthwork quantities were also compared with the quantities determined by normal field methods. The three general cross sections are shown in Figure 3.

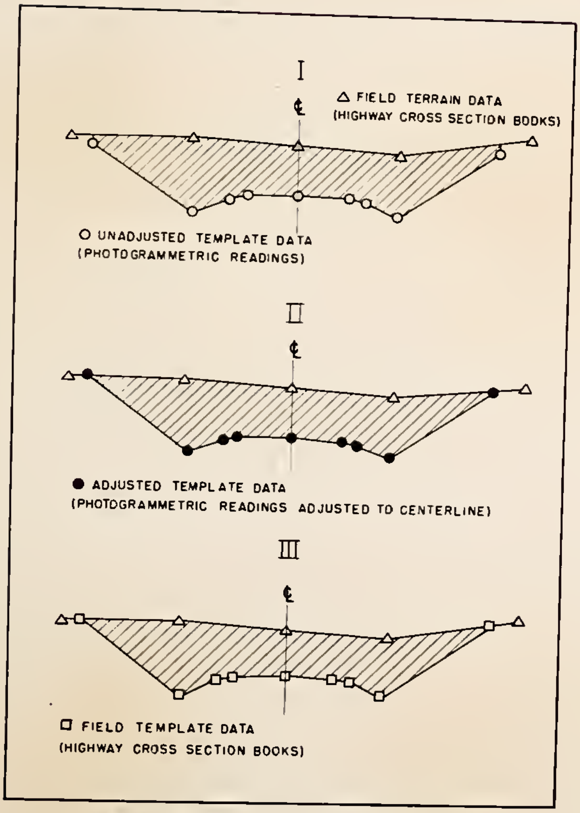
Results

Figure 4 illustrates graphically the results obtained in measuring non-earthwork quantities.

Concrete Pavement and Appurtenances

The photogrammetric measurements of these quantities were combined and compared with the field measurements in accordance with the quantity subdivisions of the construction record. A total of 21 sections were measured. No slope corrections were made because the grades were very small. The combined error in measurement was essentially zero and the error range was from 0 to -3.7%. The largest error of -18.8 square yards or -3.7% was made in determining the area at a street intersection within the urban section.

Separate comparisons (not shown on Figure 4) of eight private drive entrances also showed good agreement between the photogrammetric and field


I
℄
△ FIELD TERRAIN DATA
(HIGHWAY CROSS SECTION BOOKS)
○ UNADJUSTED TEMPLATE DATA
(PHOTOGRAMMETRIC READINGS)
II
℄
● ADJUSTED TEMPLATE DATA
(PHOTOGRAMMETRIC READINGS ADJUSTED TO CENTERLINE)
III
℄
□ FIELD TEMPLATE DATA
(HIGHWAY CROSS SECTION BOOKS)

CROSS SECTION DATA COMBINATIONS

Figure 3. General cross sectional data combinations.

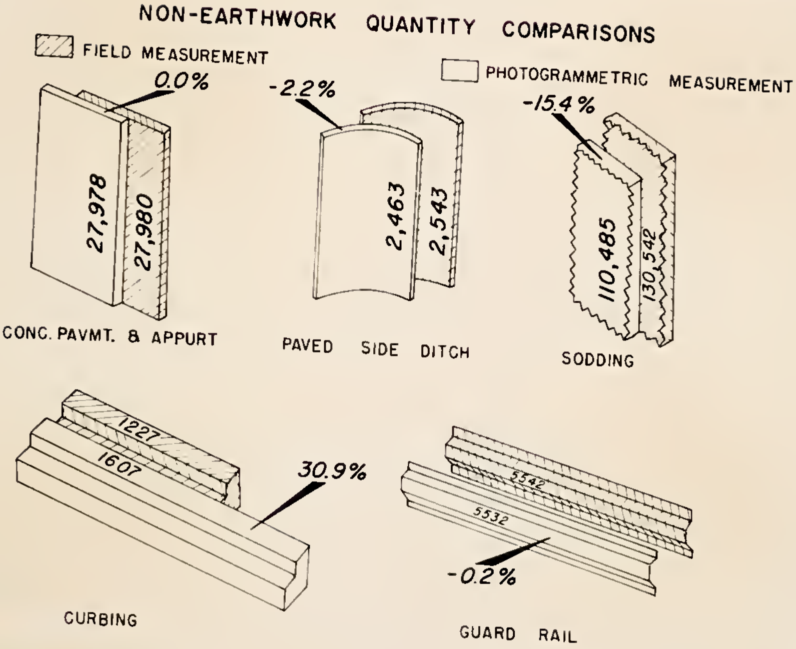


Figure 4. Results of non earthwork quantity measurements.

measurements. The combined error was a -2.2% (2.8 square yards) with the error range from 2.4 to -8.3%.

Paved Side Ditch

Measurements were made along the length of the paved side ditch. All measurements were corrected for slope. The construction record listed 15 sections of paved side ditch; but, due to unmelted snow in shadow zones, only 13 were observed and measured in the stereo model. Snow, silt and debris near the ends of the sections caused errors in identification as well as measurement. A better indication of photogrammetric measurement was obtained by omitting the two unidentified sections. The difference between photogrammetric and field quantities was then only 16 feet or 0.7 per cent. This error is deceptive since it results from the compensation of one large plus error and several small minus errors. The elimination of the large plus error caused by silt cover and snow results is a total error of 27.5 feet or 1.5 per cent. Errors in identification must be taken into account as they can be expected to occur.

Sodding

Ninety-four sections of what were considered to be sodded areas were measured and compared. Slope corrections were applied in both the longitudinal and transverse directions. Primarily errors in identification occurred because the photographs were obtained in the winter months, but a few small errors in measurement also occurred. The combined error was a -15.4% but the errors ranged from 33.1 to -60.4%. A difference of a few inches in a narrow band of sodding caused a large percentage error. This type of error was unavoidable since measurements could be made only to the nearest 0.5 of a foot.

Curbing

Poor agreement was obtained between field and photogrammetric quantities of curbing because it was impossible to distinguish between newly placed curbing and curbing which existed in the urban area prior to construction. Of the nine sections compared the combined error was 30.9% with an error range from 0 to 178%.

Guard Rail and Guide Post

Relative good agreement between the field and photogrammetric quantities was obtained in the guard rail comparisons. Eleven sections were measured for a total error of only a -0.2% (-11.0 feet) with the range in errors from 0 to -0.7%. Because of the small grades, no slope corrections were applied.

Guide posts were counted in 12 different groups. Three of the 12 groups were in error from the construction record. It was impossible to distinguish between posts placed prior to construction and new posts; therefore, the photogrammetric count gave 54 and the field count 46. Eight posts were counted in an 80-foot section in which no guide posts were listed by the construction record. In another section, two posts were counted in the stereo model for which the construction record listed only one. It may be that the construction record was in error on this count since these posts were definitely on newly constructed fill.

Earthwork Quantities

Figure 5 illustrates graphically the results in measuring earthwork quantities over the entire length of line. A total of 238 sections of earthwork were compared to include excavation, embankment and borrow. Only excavation quantities were compared to the construction record. The

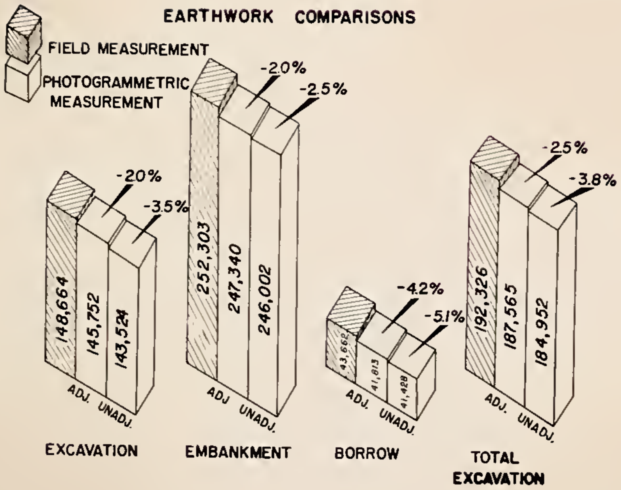


Figure 5. Earth quantity comparisons.

unadjusted quantities were compared and then a correction or adjustment was made to the center line datum as measured in the field and a comparison was made.

In excavation, the unadjusted quantities were in error from field measurements by a -3.5% and the adjusted quantities only a -2.0%. Errors ranged from 0 to incalcuible values on those sections in which the field measurements for excavation were 0. In comparison to the construction record the error for the unadjusted photogrammetric quantity was -11.3 per cent and -9.9% for the adjusted quantities. The poor comparison between the construction record and the field and photogrammetric quantities in excavation is due in part to the shortened cross sections which had to be used in many places to allow machine computation. The original field terrain data did not extend to an intercept with the photogrammetric final cross section and therefore some sections were fore shortened as shown in the top diagram in Figure 2.

Borrow pit quantities by field methods amounted to 43662 cu.yd. The unadjusted photogrammetric quantities were in error a -5.1% and the adjusted quantities a -4.2%. The errors ranged from 0 to a -24.1%.

Total excavation quantities for the unadjusted photogrammetric sections were in error from the field quantities by a -3.8% and the adjusted quantities were in error by -2.5%.

The percentage errors used in making the comparisons of earthwork quantities may be very misleading if not interpreted with any consideration for the size of the quantities involved. This is especially true of very small quantities where an error of a few yards can result in a very large percentage error. In many instances where the quantities used as a basis of comparison were zero, the resulting percentage errors were incalculable.

On the other hand, very large quantities, such as the total yardage for the entire project, may show a low percentage error and yet have a yardage error that might be considered undesirably large for pay purposes. Although the percentage error method has this inherent defect, it is the only method available for evaluating earthwork quantity comparisons.

Large percentage errors were generally associated with small earthwork quantities and, conversely, small percentage errors were generally associated with large earthwork quantities.

Seven groups of individual earthwork sections were classified according to various depths of excavations and embankments and compared with the corresponding field quantities. The selection of sections for each depth of classification was made on a relative basis by stereoscopic study of the aerial photographs for the entire length of the project. These classifications included shallow, medium, and deep cut and medium and deep fill.

The percentage errors for the various cut and fill classifications ranged from 20.5 per cent for a unadjusted quantity of shallow cut to 0.4 per cent for an unadjusted quantity of deep fill. The percentage errors generally followed an inverse relationship between the depth of the earthwork and the magnitude of the percentage error.

Statistical Analysis of Vertical Accuracy

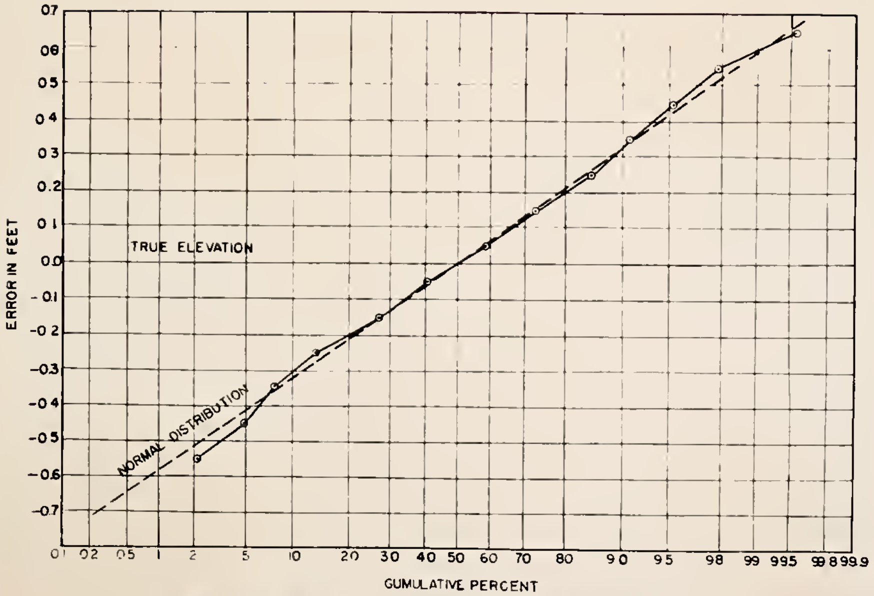
In addition to the actual comparison of final pay quantities, a statistical analysis of vertical accuracy was undertaken. Vertical accuracy is perhaps one of the most critical considerations in the determination of earthwork quantities, and it is also an important factor in the determination of slope corrections for other quantities. There

are no accepted standards by which to judge the vertical accuracy of spot elevation readings; however, the standards for vertical accuracy of contour mapping are well established and may be used as an indication of the precision with which spot elevations may be read. Certain statistical measures may also be used as an indication of precision.

The analysis was accomplished by a comparison of field centerline and base line elevations with corresponding photogrammetric elevations. The difference or "error" in the 239 elevation readings tested was treated as a random sample from the population of possible centerline and base line elevation readings. The mean, variance, standard deviation, and range of the errors were computed, and the type and distribution of errors was investigated by tests on the means and variances of the individual models and on the variances within a single model.

Figure 6 shows the frequency distribution plotted in cumulative form with the abscissa scale graduated according to the area under a normal distribution curve. The points were plotted on the basis of cumulative per cent less than the class boundaries of the errors starting with the minus values. The distribution of the elevation errors is in close agreement with the normal distribution function which is shown by a broken straight line. The importance of this distribution is emphasized by the fact that such a distribution will have a compensating affect on the total error.

Approximately 90 per cent of the points tested fall within the error range of ± .45 feet as estimated from the cumulative frequency distribution of Figure 6. According to the National Map Accuracy Standards, this would allow a usable contour interval of 0.9 foot. At a flying height of 1500 feet, the calculated C-factor is approximately 1670.



CUMULATIVE FREQUENCY DISTRIBUTION OF ELEVATION ERRORS

FIGURE 6

Figure 6. Cumulative frequency distribution of photogrammetric elevations compared with field base line and center line elevation.

The center line and base line elevation errors were processed to yield values of the mean, variance, standard deviation, and range for the entire project and for individual models and borrow pits. This subdivision of the data was used because each model was independent of all other models in the study. Differences in accuracy might beexpected since each model was separately oriented and the quality of the diapositives varied from model to model.

The means for the individual models ranged from -.144 feet to .257 feet and the standard deviation ranged from .120 feet to .314 feet. The mean for the entire project was .009 feet with a standard deviation of .252 feet. The range in errors was 1.3 feet.

Analysis of variance indicated that the mean error for the entire sample was not significantly different from zero at the five per cent level of significance but that the mean errors for most of the models were significantly different from zero. Since the range of errors would indicate that no serious blunders were made, these tests would seem to indicate that systematic errors were operative from model to model. It appears that these systematic errors compensated each other in such a manner that the mean of all errors was practically zero.

To test the precision of elevation readings within the model, repeated elevation readings were made at nine points selected at random from nine subdivisions of a single model area. Ten independent readings were made at each point. The variances were computed and analyzed by Bartlett's test. Again, the variances were found to be significantly different at the five per cent level.

The results of this test would seem to indicate that the variation in the precision of elevation readings is dependent not only upon the model

but also upon the point at which the reading is made within the model. This would tend to coincide with actual plotting experience in which it was observed that the elevations of same points were more difficult to determine than others because of the varying degree of ground cover and image clarity and the lack of contrast in tone and terrain.

Summary and Conclusions

In conducting this study, an attempt was made to approximate typical conditions and situations. It must be recognized, however, that no section of highway can be classified as typical. Variations in terrain, land use, and physical features of highways make this an impossibility. Nor can the equipment or procedures which were employed be classified as representative. A wide variety of photogrammetric plotting instruments, flying heights, and plotting procedures could be used in an undertaking of this nature. Caution must be exercised, therefore, in any generalization of the results and conclusions of this particular study.

From the experience gained in the execution of this study, it may be concluded that the photogrammetric methods and procedures described herein are applicable, with limited regard for accuracy, to the measurement of the following final pay quantities: earthwork, concrete pavement and appurtenances, paved side ditch, sodding, curbing, guard rail, and guide posts.

From the analysis and results of the accuracy comparisons of this study, it may be concluded that:

1. There was good agreement between the photogrammetric earthwork excavation quantities and the corresponding quantities computed electronically from the data in the field cross section books.

2. The photogrammetric embankment quantities also showed good agreement with the corresponding quantities computed electronically from data in the field cross section books.
3. Adjustment of the photogrammetric earthwork quantities to an accurate center line profile generally improved accuracy.
4. The relatively large errors of some of the individual sections of earthwork compensated to yield a smaller error for the total earthwork.
5. The percentage errors generally varied inversely with the depth and size of the earthwork quantity.
6. The photogrammetric measurements of concrete pavement and appurtenances were in close agreement with the construction record measurements. (The percentage error was, for all practical purposes, zero).
7. Due to the compensation of a few large errors, the photogrammetric measurements of paved side ditch showed fairly good agreement with the construction record. (The photogrammetric measurements were in error by -3.1 per cent.)
8. Poor agreement was obtained in the comparison of photogrammetric quantities of sodding with the corresponding construction record quantities. (The error was -15.4 per cent).
9. Poor agreement was obtained in the photogrammetric and construction record comparisons of curbing. (The error was 30.9 per cent.)
10. The photogrammetric measurements of guard rail were in close agreement with the construction record measurements. (The photogrammetric quantities were in error by -0.2 per cent.)

11. The photogrammetric count of guide posts did not agree well with the construction record count. (The construction record listed a total of 46 guide posts compared to the photogrammetric count of 54.)

As a corollary to the above conclusions, it may be stated that the photogrammetric techniques described herein definitely provide accuracy and reliability of a nature that would warrant their use in the location and design phases of highway construction. This is true not only for the preliminary stages but also for the final stages of location and design as well.

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2. Funk, L. L., "Adjustment of Photogrammetric Surveys", Bulletin 228, Highway Research Board, No. 688, pp. 21-27, January, 1956.
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TRAFFIC SPEED REPORT
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by

N. JOUZY

Joint
Highway
Research
Project

PURDUE UNIVERSITY
LAFAYETTE INDIANA

Progress Report

TRAFFIC SPEED REPORT NO. 69

TO: K. B. Woods, Director
Joint Highway Research Project

June 8, 1960

FROM: H. L. Michael, Assistant Director
Joint Highway Research Project

File: 8-3-3

"Traffic Speed Report No. 69" authored by Neddy Jouzy, Graduate Research Assistant on our staff is attached. This progress report on the continuing study of speed trends has been conducted under the supervision of Professor H. L. Michael.

The results of this study indicate that speeds of both passenger cars and trucks were slightly less during the period of the study than they were in August 1959.

Copies of this report after acceptance by the Board will in addition to the normal distribution also be forwarded to the Indiana State Police, Office of Traffic Safety, the Traffic Engineering Department of the State Highway Department, and the Bureau of Public Roads.

The report is submitted for the record and for release for distribution.

Respectfully submitted,

Harold L Michael

Harold L. Michael, Secretary

HLM:kmc

Attachment

cc: F. L. Ashbaucher
J. R. Cooper
W. L. Dolch
W. H. Goetz
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F. F. Havey
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R. D. Miles
R. E. Mills
C. E. Vogelgesang
J. L. Waling
J. E. Wilson
E. J. Yoder

Progress Report

Traffic Speed Report No. 69

by

Neddy Jouzy
Graduate Research Assistant

Joint Highway Research Project
File: 8-3-3
Project: C-36-10C

Purdue University
Lafayette, Indiana

June 8, 1960

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U.S. 52-1.0 MILES WEST OF KLONDIKE III

U.S. 52-2.2 MILES NORTHWEST OF TEMPLETON IV

U.S. 31-7.2 MILES NORTH OF PERRYSBURG V

S.R. 25- .7 MILES SOUTH OF AMERICUS VI

U.S. 41-1.0 MILES NORTH OF BOSWELL VII

FIGURES — FIGURE NO.

PERCENTILE SPEED CURVES

U.S. 52-1.0 MILES SOUTH OF S. JCT. S.R. 28 1

U.S. 52-1.0 MILES WEST OF KLONDIKE 2

U.S. 52-2.2 MILES NORTHWEST OF TEMPLETON 3

U.S. 31-7.2 MILES NORTH OF PERRYSBURG 4

S.R. 25- .7 MILES SOUTH OF AMERICUS 5

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TRAFFIC SPEED REPORT NO. 69

This report covers spot speed observations made during the months of March and April 1960. All observations were made of free moving vehicles on level tangent sections of rural highways. The locations of the spot stations were the same as for previous studies and are as follows:

1. U. S. 52 - 1.0 mile south of south junction of S. R. 28 - (Dual Lanes)
2. U. S. 52 - 1.0 mile west of Klondike (Dual Lanes)
3. U. S. 52 - 2.2 miles northwest of Templeton (2 lanes)
4. U. S. 31 - 7.2 miles north of Perrysburg (2 lanes)
5. S. R. 25 - 0.7 mile south of Americus (2 lanes)
6. U. S. 41 - 1.0 mile north of Boswell (2 lanes)

An Electromatic Radar Speed Meter was used to collect the data for this study. The meter was concealed as part of a rural mailbox and placed close by to the edge of the pavement. It was directed along the highway at a small angle with respect to the direction of traffic so that it was not necessary to make an angle correction to the readings.

The observers concealed themselves from traffic as much as local conditions permitted and it is thus believed that the speeds of the observed vehicles were not influenced by the observer or the equipment.

The radar equipment used will not operate properly if the voltage varies more than minus 1/2 volt or plus 1 volt from the optimum twelve volts. The voltage was therefore checked periodically in the field and maintained within the desired range. Also, correct calibration was maintained by periodically checking it with a 60 mile per hour tuning fork.

A summary of the results of this study, as well as of the last seven studies, is given in Table I. Indiana State law limits the speed of passenger cars and trucks under 5,000 pounds (GVW) to 65 miles per hour. The speed limit of trucks over 5,000 pounds (GVW) is 50 miles per hour on all highways except that on four-lane highways, which have a median strip of at least twenty feet in width, the speed limit is 55 miles per hour. This new speed limit for trucks has been in effect since about May 1959. However, some, but not all, truck speed limit signs have been posted along the state highways to reflect this change. It can be assumed though, that by now a large portion of the truck drivers are aware of this speed limit change.

Truck speeds are shown in the tables for three groups: light trucks, heavy trucks, and all trucks. Also, it is noted whether the highway is 2-lane or 4-lane. All 4-lane observation stations were on highways with a median of greater than twenty feet. Since weights were not taken in this study, light trucks (less than 5,000 pounds) were considered to be only panel trucks and pick-up trucks. All other types of trucks were classified as heavy (over 5,000 pounds).

The passenger car data were also classified into three groups: Indiana, out of state, and all passenger cars. The classification was determined by observing the license plate on each passenger car passing the observation station.

So that an easy comparison can be made between the results of the present study and the last previous study, the results from each speed station for both studies are tabulated in Tables II through VII.

Average and 85th percentile speeds were computed for each vehicle group from the sample of vehicles taken at each study site. A comparison

is then made with the speeds of the last study in July and August of 1959, and the differences in speeds are briefly discussed.

The average speed for all passenger cars decreased by 1.4 miles per hour since the last study (August 1959) while average speeds for all trucks decreased 1.1 miles per hour. Indiana passenger cars decreased their average speed on 2-lane highways by 2.3 miles per hour while their average speed on 4-lane highways decreased by 0.3 miles per hour. Out of state passenger cars decreased their average speed on 2-lane highways by 0.5 miles per hour while their average speed on 4-lane highways decreased by 0.9 miles per hour.

The average speed for light trucks decreased by 1.2 miles per hour on 2-lane highways and increased 2.4 miles per hour on 4-lane highways.

For heavy trucks the average speed decreased 0.8 miles per hour on 2-lane highways and decreased 5.8 miles per hour on 4-lane highways.

The 85th percentile speed for all passenger cars decreased by 0.5 miles per hour.

Trend information on the average speed of passenger cars and trucks is shown in Table 1 and Figure 7 and 8. Table 1 is a summary of Spot Speed Observations on Indiana Highways for the last eight studies since February 1956. This summary of Spot Speed lists the observations for Two-lane Highways, Four-lane Highways and All Highways for both passenger cars and trucks.

Figure 7 is a graph showing rural speed trends from 1942-1960. The speed trends shown are for passenger cars, light trucks and heavy trucks.

Figure 8 is a graph showing trends in percentile speeds and speed differential from 1949 - 1960. The percentile speeds shown are for both

passenger cars and heavy trucks. The speed differential is the difference between the 85th percentile of passenger cars and 15th percentile of heavy trucks.

TABLE I

SUMMARY OF SPOT SPEED OBSERVATIONS ON INDIANA HIGHWAYS

(Free-Moving Vehicles on Level, Tangent Sections)

		Passenger Cars				Trucks		
		Ind Mean	Non-Ind Mean	All Mean	All 85 per	Light Mean	Heavy Mean	All Mean
Two-Lane Highways	Feb. '56	54.9	58.0	55.9	63.2	47.1	43.2	44.4
	Aug. '56	55.0	56.3	55.5	63.4	50.6	45.5	46.6
	May '57	55.6	59.1	56.9	64.0	50.2	44.8	46.1
	Aug. '57	55.5	56.7	55.9	62.1	51.7	45.8	47.3
	Aug. '58	54.3	56.3	55.0	61.7	53.0	46.3	47.8
	Mar. '59	55.5	57.7	56.1	61.9	50.4	45.4	46.3
	Aug. '59	55.6	55.3	55.9	63.2	49.7	46.7	48.0
	Mar. '60	53.27	54.8	53.7	61.2	48.5	45.9	46.6
Four-lane Highways	Feb. '56	58.1	60.1	58.7	65.7	47.8	45.2	45.8
	Aug. '56	57.4	58.8	58.2	66.8	49.6	46.0	47.4
	May '57	59.9	63.6	61.0	69.0	52.2	46.0	47.9
	Aug. '57	57.5	59.9	58.5	64.8	52.0	46.6	47.6
	Aug. '58	58.0	59.6	58.7	65.0	54.3	49.0	50.0
	Mar. '59	58.2	61.7	59.0	64.9	53.6	47.0	48.4
	Aug. '59	58.4	60.1	59.1	64.5	50.0	53.1	49.5
	Mar. '60	58.1	59.2	58.4	65.0	52.4	47.3	48.4
All Highways	Feb. '56	56.0	58.6	56.8	63.8	47.3	44.0	44.9
	Aug. '56	55.7	57.3	56.4	64.5	50.2	45.6	46.9
	May '57	57.2	60.3	58.3	66.0	50.9	45.2	46.6
	Aug. '57	56.2	58.3	56.9	63.2	51.8	46.1	47.4
	Aug. '58	55.7	57.9	56.5	63.1	53.4	47.5	48.7
	Mar. '59	56.6	59.0	57.2	63.1	51.5	45.9	47.0
	Aug. '59	56.5	58.4	57.4	63.5	50.6	47.8	48.6
	Mar. '60	55.6	57.1	56.0	63.0	50.2	46.5	47.5

TABLE II SPEED DATA

Station 1 Mile south of south junction of US 52 & SR 28

Surface 4 lane divided - 24' bituminous concrete Weather Cloudy - windy

This Observation
Date April 3; April 6; April 8, 1960
Time 2:15 - 2:55 PM; 1:35 - 3:30 PM; 2:45 - 4 PM

Last Previous Observation (Speed Report No. 67)
Date Aug. 3, 1959
Time 8:50 - 10:50 AM

			PASSENGER CARS						TRUCKS						BUSSES	
			All		Indiana		Non-Indiana		All		Less than 5000 pounds		5000 pounds or more		All	
		OBSERVATION	Last	Present	Last	Present	Last	Present	Last	Present	Last	Present	Last	Present	Last	Present
All Vehicles		No. of Vehicles Obs.	455	524	218	339	237	185	82	159	12	33	70	126	—	—
		Ave. Speed (m.p.h.)	60.4	60.3	59.7	60.2	61.1	60.5	50.4	48.8	55.1	53.6	49.6	47.5	—	—
	% Equaling or Exceeding	45 m.p.h.	—	—	—	—	—	—	86.6	68.0	91.7	79	85.7	65.0	—	—
		50 m.p.h.	96.3	96.0	93.6	95.5	98.7	96.0	54.8	50.0	83.3	70	55.7	44.0	—	—
		55 m.p.h.	85.3	84.0	80.1	81.4	89.5	88.0	20.7	16.0	66.7	36	12.9	11.0	—	—
		60 m.p.h.	61.3	61.0	57.3	59.0	65.0	65.0	6.1	7.0	25.0	30	2.9	1.0	—	—
		65 m.p.h.	24.8	19.0	22.0	20.0	27.4	18.0	0.0	3.0	0.0	15	0.0	0.0	—	—
		70 m.p.h.	5.3	6.0	5.0	7.0	5.5	4.0	0.0	2.5	0.0	12	0.0	0.0	—	—
		75 m.p.h.	1.3	2.0	1.4	2.0	1.3	2.0	0.0	0.0	0.0	0.0	0.0	0.0	—	—
BOUND		No. of Vehicles Obs.	239	319	110	216	129	103	44	80	5	23	39	57	—	—
		Ave. Speed (m.p.h.)	60.7	60.2	59.4	60.0	61.9	60.5	49.7	50.7	59.6	55.6	48.4	48.7	—	—
		Max. Speed (m.p.h.)	—	—	73	84	76	80	—	—	64	74	58	60	—	—
		State or Type	—	—	—	—	—	—	—	—	—	—	—	—	—	—
		Min. Speed (m.p.h.)	—	—	40	40	43	40	—	—	53	30	39	33	—	—
		State or Type	—	—	—	—	—	—	—	—	—	—	—	—	—	—
BOUND		No. of Vehicles Obs.	216	205	108	123	108	82	38	79	7	10	31	69	—	—
		Ave. Speed (m.p.h.)	60.1	60.5	59.9	60.4	60.3	60.5	51.3	46.9	51.9	49.2	51.1	46.5	—	—
		Max. Speed (m.p.h.)	—	—	88	80	76	72	—	—	58	72	60	56	—	—
		State or Type	—	—	—	—	—	—	—	—	—	—	—	—	—	—
		Min. Speed (m.p.h.)	—	—	34	44	42	38	—	—	36	36	38	34	—	—
		State or Type	—	—	—	—	—	—	—	—	—	—	—	—	—	—

TABLE III SPEED DATA

Station 1.0 Mile west of Klondike on US 52

Surface 4 Lane Divided 24' Portland Cement Concrete Weather Cloudy

This Observation
Date March 21, 1960 and March 23, 1960
Time 1:35 - 3:45 PM 1:30 - 3:30 PM

Last Previous Observation (Speed Report No. 67)
Date July 10, 1959
Time 2:45 - 5:10 PM

			PASSENGER CARS						TRUCKS						BUSSES	
			All		Indiana		Non-Indiana		All		Less than 5000 pounds		5000 pounds or more		All	
		OBSERVATION	Last	Present	Last	Present	Last	Present	Last	Present	Last	Present	Last	Present	Last	Present
All Vehicles		No. of Vehicles Obs.	385	486	228	362	157	124	82	178	15	39	67	139	—	—
		Ave. Speed (m.p.h.)	57.8	56.4	57.2	56.1	58.6	57.2	49.8	48.0	52.0	51.4	49.3	47.1	—	—
	% Equaling or Exceeding	45 m.p.h.	—	—	—	—	—	—	78.0	68.0	80.0	79.0	77.6	65.0	—	—
		50 m.p.h.	89.8	86.0	89.9	85.0	89.8	89.0	58.5	43.0	66.7	47.0	56.7	42.0	—	—
		55 m.p.h.	71.7	60.0	66.2	57.0	79.6	68.0	19.5	9.0	33.3	16.0	16.4	7.0	—	—
		60 m.p.h.	47.3	38.0	44.7	36.0	51.0	44.0	8.5	3.0	13.3	11.0	7.4	1.0	—	—
		65 m.p.h.	13.5	10.0	11.8	10.0	15.9	11.0	3.7	0.0	13.3	0.0	1.5	0.0	—	—
		70 m.p.h.	2.9	4.0	2.2	4.0	3.8	4.0	1.2	0.0	6.7	0.0	0.0	0.0	—	—
		75 m.p.h.	0.5	0.0	0.4	1.0	0.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	—	—
BOUND		No. of Vehicles Obs.	192	251	124	189	68	62	42	82	9	20	33	62	—	—
		Ave. Speed (m.p.h.)	59.5	58.1	58.6	57.6	61.3	59.7	51.6	50.0	52.3	53.2	51.4	49.0	—	—
		Max. Speed (m.p.h.)	—	—	70	94	76	72	—	—	66	76	62	60	—	—
		State or Type	—	—	—	—	—	—	—	—	—	—	—	—	—	—
		Min. Speed (m.p.h.)	—	—	42	32	44	44	—	—	30	40	25	34	—	—
		State or Type	—	—	—	—	—	—	—	—	—	—	—	—	—	—
BOUND		No. of Vehicles Obs.	193	235	104	173	89	62	40	96	6	19	34	77	—	—
		Ave. Speed (m.p.h.)	56.0	54.5	55.4	54.4	56.5	54.8	47.9	46.3	51.5	49.5	47.3	45.5	—	—
		Max. Speed (m.p.h.)	—	—	70	74	70	68	—	—	72	60	66	58	—	—
		State or Type	—	—	—	—	—	—	—	—	—	—	—	—	—	—
		Min. Speed (m.p.h.)	—	—	30	34	36	36	—	—	38	40	38	28	—	—
		State or Type	—	—	—	—	—	—	—	—	—	—	—	—	—	—

TABLE IV SPEED DATA

Station 2.0 Miles northwest Templeton on US 52

Surface 2 Lane 22' Bituminous Weather Cloudy

This Observation
Date March 25, 1960
Time 3:30 - 5:00 PM

Last Previous Observation (Speed Report No. 67)
Date July 17, 1959
Time 3:00 - 4:30 PM

			PASSENGER CARS						TRUCKS						BUSSES	
			All		Indiana		Non-Indiana		All		Less than 5000 pounds		5000 pounds or more		All	
		OBSERVATION	Last	Present	Last	Present	Last	Present	Last	Present	Last	Present	Last	Present	Last	Present
All Vehicles		No. of Vehicles Obs.	222	157	99	64	123	93	69	41	7	4	62	37	—	—
		Ave. Speed (m.p.h.)	56.5	56.7	56.3	57.0	56.7	56.4	48.3	47.6	52.4	49.8	47.9	47.3	—	—
	% Equaling or Exceeding	45 m.p.h.	—	—	—	—	—	—	84.1	78.1	100	75.0	82.3	78.0	—	—
		50 m.p.h.	86.0	92.0	82.8	92.2	88.6	91.4	47.8	48.8	85.7	75.0	43.5	46.0	—	—
		55 m.p.h.	63.1	61.0	59.6	64.0	65.9	58.0	7.2	5.0	42.9	25.0	3.2	3.0	—	—
		60 m.p.h.	35.6	42.0	38.4	33.0	33.3	32.0	1.4	0.0	0.0	0.0	1.6	0.0	—	—
		65 m.p.h.	10.8	9.0	11.1	9.0	10.6	9.0	0.0	0.0	0.0	0.0	0.0	0.0	—	—
		70 m.p.h.	4.1	3.0	4.0	5.0	4.1	2.0	0.0	0.0	0.0	0.0	0.0	0.0	—	—
		75 m.p.h.	0.5	0.0	1.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	—	—
BOUND		No. of Vehicles Obs.	155	96	64	38	91	58	38	23	4	2	34	21	—	—
		Ave. Speed (m.p.h.)	56.7	57.4	56.2	56.6	57.0	58.0	48	47.3	53.5	50	47.4	47.0	—	—
		Max. Speed (m.p.h.)	—	—	74	72	70	72	—	—	56	50	54	55	—	—
		State or Type	—	—	—	—	—	—	—	—	—	—	—	—	—	—
		Min. Speed (m.p.h.)	—	—	28	46	39	48	—	—	50	50	28	38	—	—
		State or Type	—	—	—	—	—	—	—	—	—	—	—	—	—	—
BOUND		No. of Vehicles Obs.	67	61	35	26	32	35	26	18	3	2	23	16	—	—
		Ave. Speed (m.p.h.)	56.3	55.4	56.5	57.7	56.0	53.7	48.7	47.9	51.0	49.5	48.5	47.7	—	—
		Max. Speed (m.p.h.)	—	—	78	70	70	72	—	—	56	55	60	53	—	—
		State or Type	—	—	—	—	—	—	—	—	—	—	—	—	—	—
		Min. Speed (m.p.h.)	—	—	38	40	42	42	—	—	46	44	38	40	—	—
		State or Type	—	—	—	—	—	—	—	—	—	—	—	—	—	—

TABLE V SPEED DATA

Station 7.8 Miles N. of Perrysburg on US 31

Surface 2 Lane 22' Bituminous

This Observation
Date March 29, 1960
Time 12:45 - 4:15 PM

Weather Cloudy

Last Previous Observation (Speed Report No. 67)
Date Aug. 10, 1959
Time 10:10 AM - 12:40 PM

		PASSENGER CARS						TRUCKS						BUSSES	
		All		Indiana		Non-Indiana		All		Less than 5000 pounds		5000 pounds or more		All	
	OBSERVATION	Last	Present	Last	Present	Last	Present	Last	Present	Last	Present	Last	Present	Last	Present
All Vehicles	No. of Vehicles Obs.	330	299	250	246	80	53	86	102	27	26	59	76	—	—
	Ave. Speed (m.p.h.)	56.0	52.8	56.1	52.5	55.7	54.3	48.8	47.0	48.9	50.2	48.8	45.9	—	—
% Equaling or Exceeding	45 m.p.h.	—	—	—	—	—	—	74.4	59.8	49.3	96.0	81.4	92.0	—	—
	50 m.p.h.	83.0	71.0	83.2	66.5	82.5	81.4	48.8	40.2	44.4	73.0	50.8	55.0	—	—
	55 m.p.h.	61.2	41.0	63.2	40.7	55.0	37.2	15.1	8.8	22.2	62.0	11.9	33.0	—	—
	60 m.p.h.	33.0	23.0	33.2	23.0	32.5	9.3	5.8	3.0	11.1	27.0	3.4	3.0	—	—
	65 m.p.h.	9.1	5.0	9.6	6.4	7.5	2.3	3.5	0.0	7.4	8.0	1.7	1.0	—	—
	70 m.p.h.	3.3	2.0	3.2	2.5	3.8	0.0	0.0	0.0	7.4	0.0	0.0	0.0	—	—
	75 m.p.h.	1.2	0.0	1.2	0.0	0.0	0.0	0.0	0.0	3.7	0.0	0.0	0.0	—	—
BOUND	No. of Vehicles Obs.	162	156	130	128	32	28	47	54	17	14	30	40	—	—
	Ave. Speed (m.p.h.)	57.7	54.8	57.1	54.7	56.9	55.4	50.3	49.3	50.7	51.9	50.2	48.4	—	—
	Max. Speed (m.p.h.)	—	—	90	72	72	66	—	—	76	58	62	56	—	—
	State or Type	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	Min. Speed (m.p.h.)	—	—	32	34	48	40	—	—	33	40	38	42	—	—
	State or Type	—	—	—	—	—	—	—	—	—	—	—	—	—	—
BOUND	No. of Vehicles Obs.	168	143	120	118	48	25	39	48	10	12	29	36	—	—
	Ave. Speed (m.p.h.)	54.9	50.6	55.0	50.1	54.9	53.0	47.2	44.4	45.8	48.2	47.6	43.2	—	—
	Max. Speed (m.p.h.)	—	—	78	70	80	64	—	—	54	60	66	60	—	—
	State or Type	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	Min. Speed (m.p.h.)	—	—	31	30	34	36	—	—	38	32	32	26	—	—
	State or Type	—	—	—	—	—	—	—	—	—	—	—	—	—	—

TABLE VI SPEED DATA

Station 0.7 miles W. Americus on S. R. 25

Surface 2 lane 22' Bituminous

Weather Fair and Sunny

This Observation
Date April 1, 1960
Time 2:45 - 4:20 PM

Last Previous Observation (Speed Report No. 67)
Date July 16, 1960
Time 2:35 - 5:05 PM

		PASSENGER CARS						TRUCKS						BUSSES	
		All		Indiana		Non-Indiana		All		Less than 5000 pounds		5000 pounds or more		All	
	OBSERVATION	Last	Present	Last	Present	Last	Present	Last	Present	Last	Present	Last	Present	Last	Present
All Vehicles	No. of Vehicles Obs.	325	344	284	315	41	29	69	96	23	33	46	63	—	—
	Ave. Speed (m.p.h.)	54.8	53.9	54.6	53.8	55.8	55.0	48.8	46.7	49.5	48.5	48.0	45.7	—	—
% Equaling or Exceeding	45 m.p.h.	—	—	—	—	—	—	73.9	65.0	73.9	76.0	73.9	59.0	—	—
	50 m.p.h.	78.8	77.0	78.9	76.5	78.0	83.0	42.0	30.0	52.2	39.0	37.0	25.0	—	—
	55 m.p.h.	51.7	49.0	49.6	48.0	65.9	52.0	20.3	6.0	34.8	9.0	13.0	5.0	—	—
	60 m.p.h.	30.5	21.0	29.6	20.0	36.6	24.0	5.8	3.0	13.0	9.0	2.2	0.0	—	—
	65 m.p.h.	8.3	6.0	8.5	6.0	7.3	3.0	2.9	0.0	4.3	0.0	2.2	0.0	—	—
	70 m.p.h.	3.4	2.0	3.9	2.0	0.0	0.0	1.4	0.0	0.0	0.0	2.2	0.0	—	—
	75 m.p.h.	0.6	1.0	0.7	1.0	0.0	0.0	1.4	0.0	0.0	0.0	2.2	0.0	—	—
BOUND	No. of Vehicles Obs.	136	150	117	134	19	16	33	49	8	9	25	40	—	—
	Ave. Speed (m.p.h.)	54.0	54.3	53.6	54.1	56.7	56.1	46.0	45.0	48.0	45.4	45.9	44.9	—	—
	Max. Speed (m.p.h.)	—	—	70	79	68	66	—	—	60	53	56	56	—	—
	State or Type	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	Min. Speed (m.p.h.)	—	—	32	34	40	42	—	—	40	36	36	32	—	—
	State or Type	—	—	—	—	—	—	—	—	—	—	—	—	—	—
BOUND	No. of Vehicles Obs.	189	194	167	181	22	13	36	47	15	24	21	23	—	—
	Ave. Speed (m.p.h.)	55.3	53.7	55.4	53.7	55.0	53.6	51.3	48.4	51.8	49.6	51.0	47.1	—	—
	Max. Speed (m.p.h.)	—	—	88	76	66	63	—	—	65	65	76	55	—	—
	State or Type	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	Min. Speed (m.p.h.)	—	—	40	36	40	42	—	—	40	37	40	40	—	—
	State or Type	—	—	—	—	—	—	—	—	—	—	—	—	—	—

TABLE VII SPEED DATA

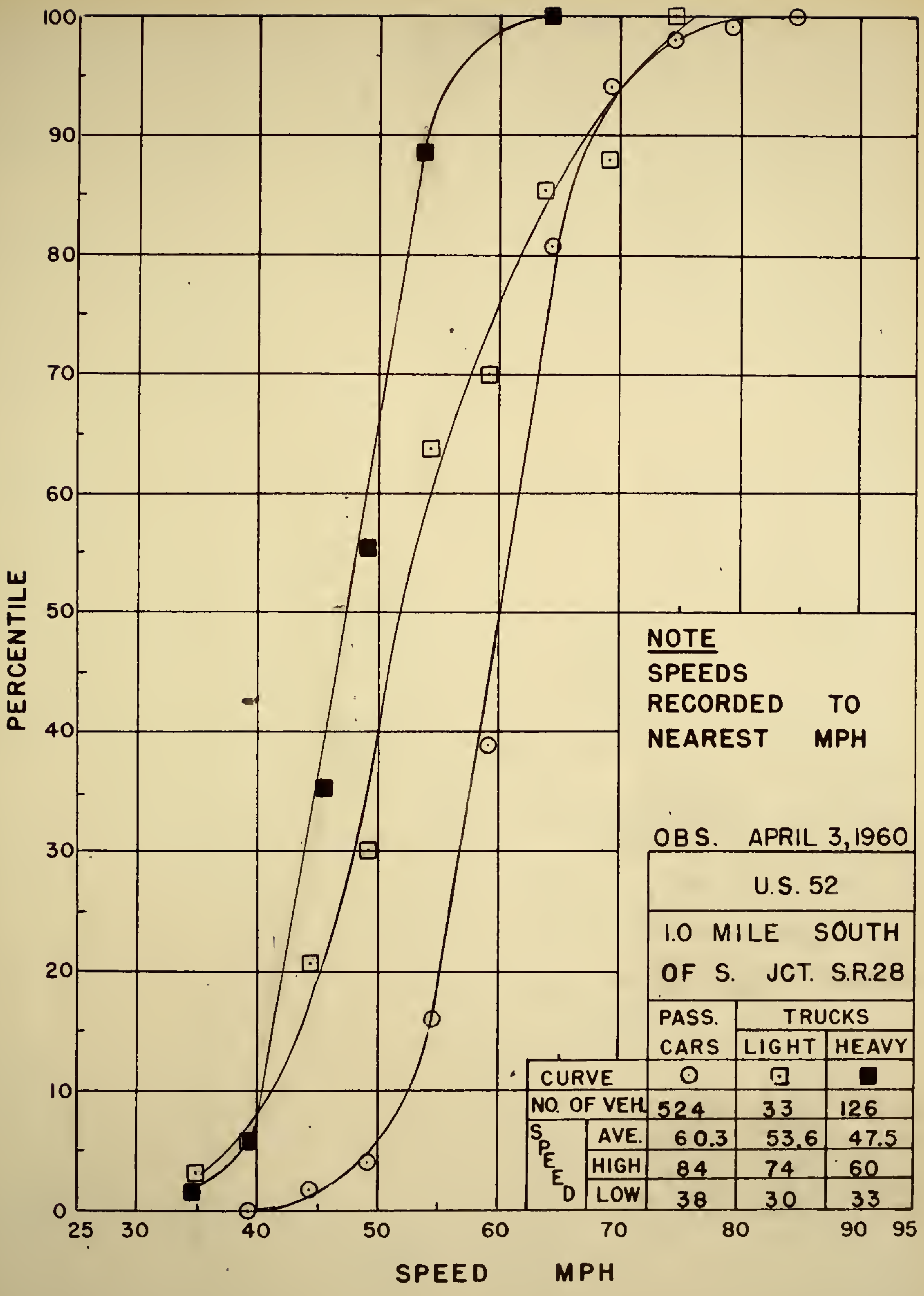
Station 1.0 mile north of Boswell on US 41

Surface 22' Portland Cement Concrete 2-lane Weather Warm and Sunny

This Observation
Date April 12, 1960
Time 2:45 - 5:00 PM

Last Previous Observation (Speed Report No. 67)
Date Sept. 1, 1959
Time 9:00 - 11:45 AM

		PASSENGER CARS						TRUCKS						BUSSES	
		All		Indiana		Non-Indiana		All		Less than 5000 pounds		5000 pounds or more		All	
OBSERVATION		Last	Present	Last	Present	Last	Present	Last	Present	Last	Present	Last	Present	Last	Present
All Vehicles	No. of Vehicles Obs.	259	255	124	149	135	106	87	104	24	31	63	73	—	—
	Ave. Speed (m.p.h.)	57.0	52.6	56.4	51.8	57.6	53.7	46.3	45.8	50.6	47.0	44.6	45.3	—	—
% Equaling or Exceeding	45 m.p.h.	—	—	—	—	—	—	59.8	54.0	83.3	48	50.8	56	—	—
	50 m.p.h.	84.6	71	83.1	67.0	85.9	76.5	33.3	31.0	54.2	35	25.4	29	—	—
	55 m.p.h.	66.0	42	63.5	38.0	68.9	48.0	8.0	8.0	16.7	19	4.8	3	—	—
	60 m.p.h.	43.2	18	41.1	15.0	45.2	23.0	3.4	4.0	12.5	13	0.0	0.0	—	—
	65 m.p.h.	13.1	4	11.2	3.0	14.8	5.0	2.2	2.0	8.3	6	0.0	0.0	—	—
	70 m.p.h.	4.2	2	4.0	1.0	4.4	2.0	0.0	2.0	0.0	6	0.0	0.0	—	—
	75 m.p.h.	0.4	0.0	0.8	0.0	0.0	0.0	0.0	1.0	0.0	3	0.0	0.0	—	—
BOUND	No. of Vehicles Obs.	129	148	67	76	62	72	37	47	11	15	26	32	—	—
	Ave. Speed (m.p.h.)	57.3	53.5	56.6	52.5	58.1	54.6	46	44.6	52	47.2	43.4	43.5	—	—
	Max. Speed (m.p.h.)	—	—	77	70	74	72	—	—	69	76	56	58	—	—
	State or Type	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	Min. Speed (m.p.h.)	—	—	35	34	40	38	—	—	40	36	30	30	—	—
	State or Type	—	—	—	—	—	—	—	—	—	—	—	—	—	—
BOUND	No. of Vehicles Obs.	130	107	57	73	73	34	50	57	13	16	37	41	—	—
	Ave. Speed (m.p.h.)	56.8	51.3	56.2	51.1	44	51.7	46.5	46.8	49.5	46.9	45.5	46.7	—	—
	Max. Speed (m.p.h.)	—	—	70	70	74	64	—	—	60	60	55	56	—	—
	State or Type	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	Min. Speed (m.p.h.)	—	—	36	32	40	38	—	—	39	36	34	34	—	—
	State or Type	—	—	—	—	—	—	—	—	—	—	—	—	—	—



NOTE
SPEEDS RECORDED TO NEAREST MPH

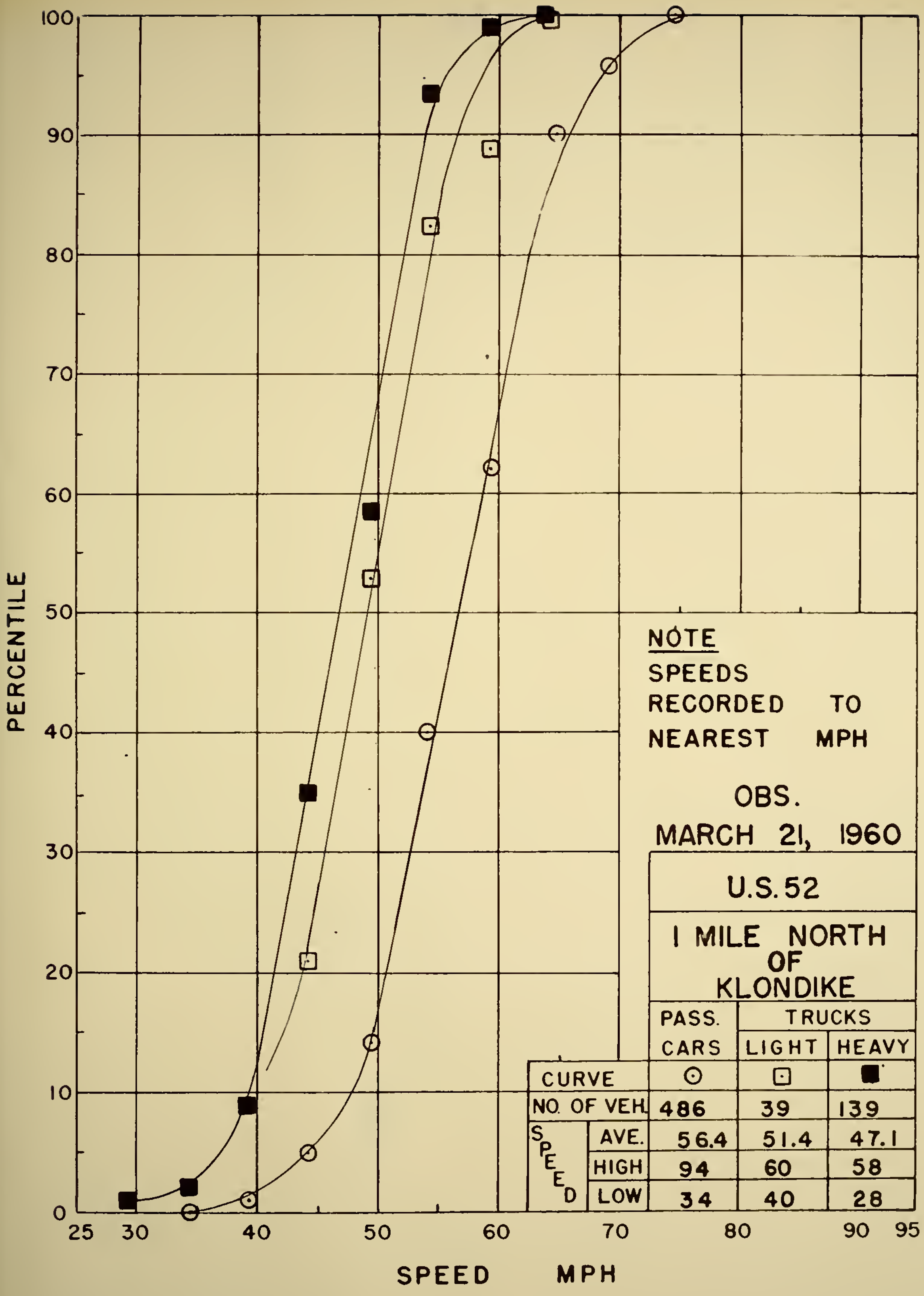
OBS. APRIL 3, 1960

U.S. 52

1.0 MILE SOUTH OF S. JCT. S.R.28

		PASS. CARS	TRUCKS LIGHT	TRUCKS HEAVY
CURVE		⊙	⊡	■
NO. OF VEH.		524	33	126
SPEED	AVE.	60.3	53.6	47.5
	HIGH	84	74	60
	LOW	38	30	33

FIGURE 1



NOTE
SPEEDS RECORDED TO NEAREST MPH

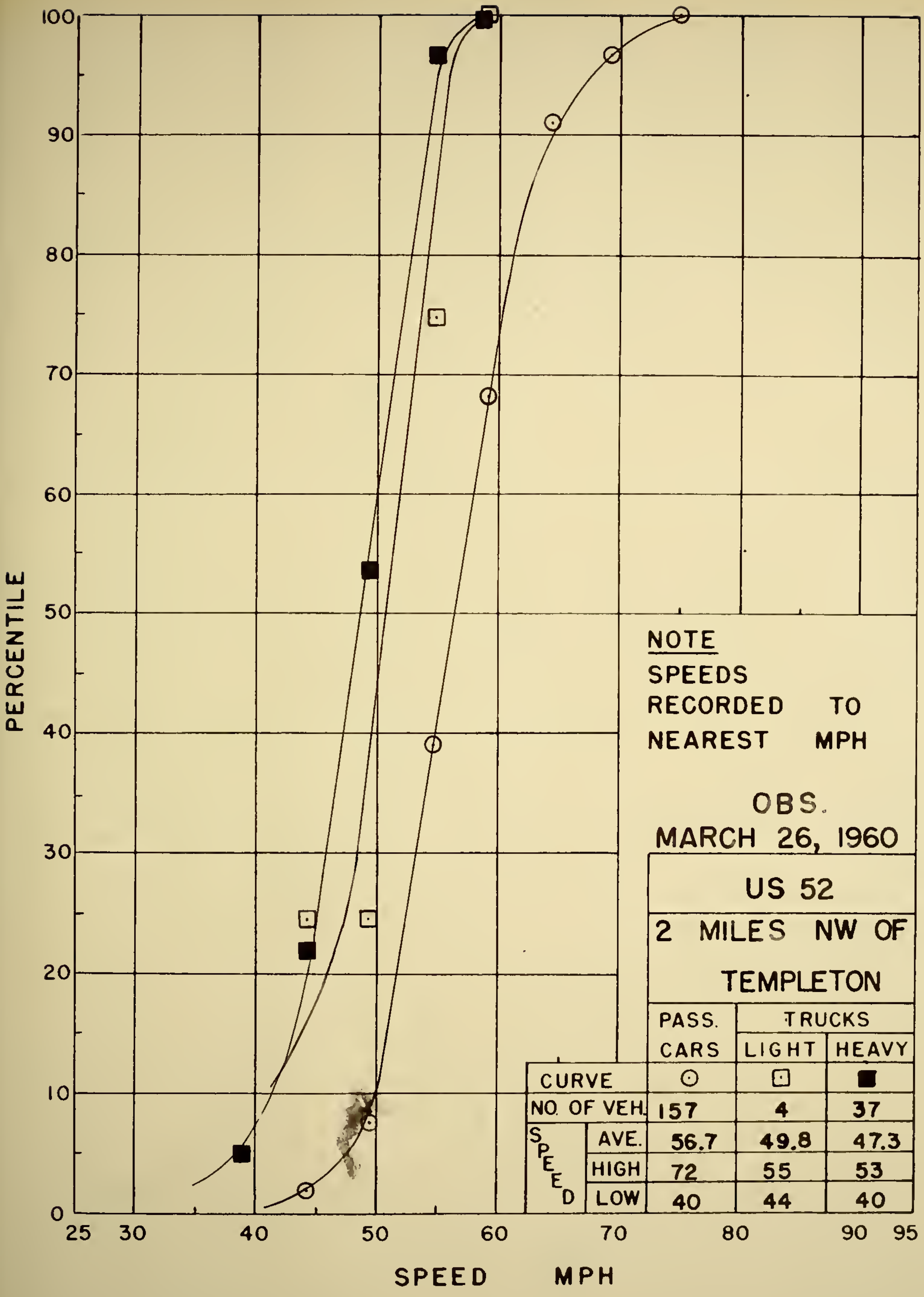
OBS.
MARCH 21, 1960

U.S. 52

I MILE NORTH OF KLONDIKE

		PASS. CARS	TRUCKS	
			LIGHT	HEAVY
CURVE		⊙	⊡	■
NO. OF VEH.		486	39	139
SPEED	AVE.	56.4	51.4	47.1
	HIGH	94	60	58
	LOW	34	40	28

FIGURE 2



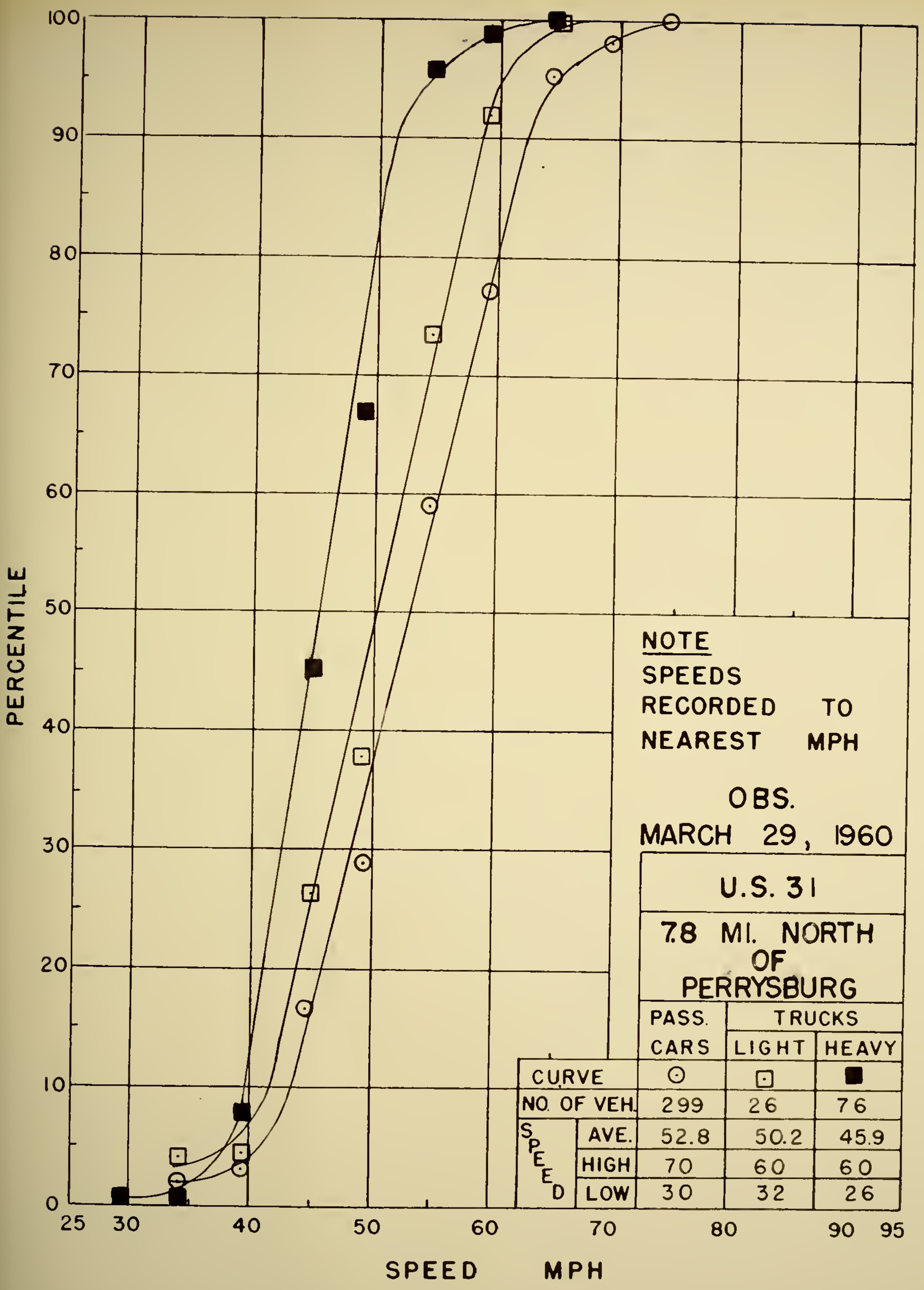
NOTE
SPEEDS
RECORDED TO
NEAREST MPH

OBS.
MARCH 26, 1960

US 52
2 MILES NW OF
TEMPLETON

		PASS. CARS	TRUCKS LIGHT	TRUCKS HEAVY
CURVE		⊙	⊡	■
NO. OF VEH.		157	4	37
SPEED	AVE.	56.7	49.8	47.3
	HIGH	72	55	53
	LOW	40	44	40

FIGURE 3



NOTE

SPEEDS RECORDED TO NEAREST MPH

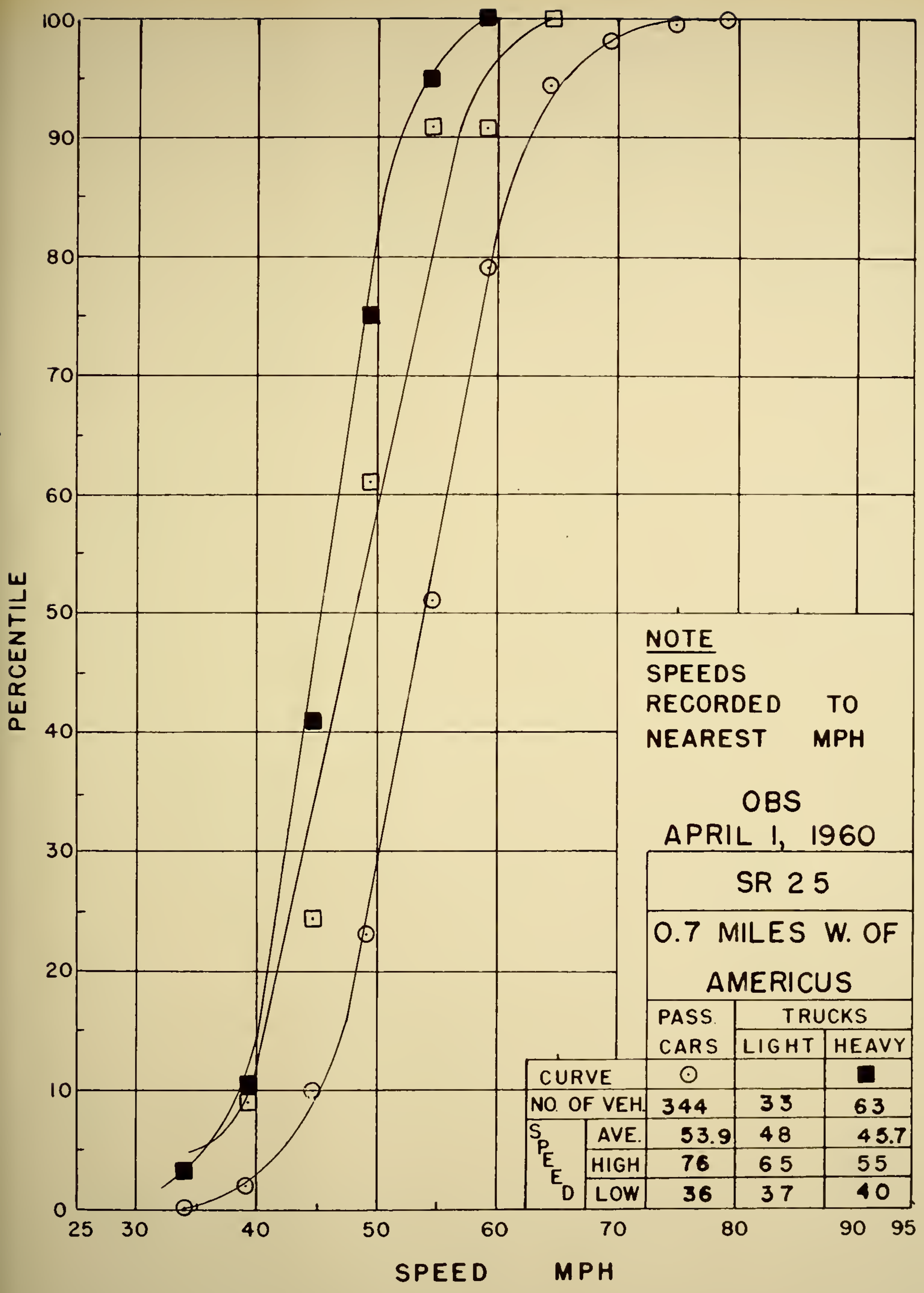
OBS.
MARCH 29, 1960

U.S. 31

7.8 MI. NORTH OF PERRYSBURG

		PASS.	TRUCKS	
		CARS	LIGHT	HEAVY
CURVE		⊙	⊡	■
NO. OF VEH.		299	26	76
SPEED	AVE.	52.8	50.2	45.9
	HIGH	70	60	60
	LOW	30	32	26

FIGURE 4



NOTE
SPEEDS RECORDED TO NEAREST MPH

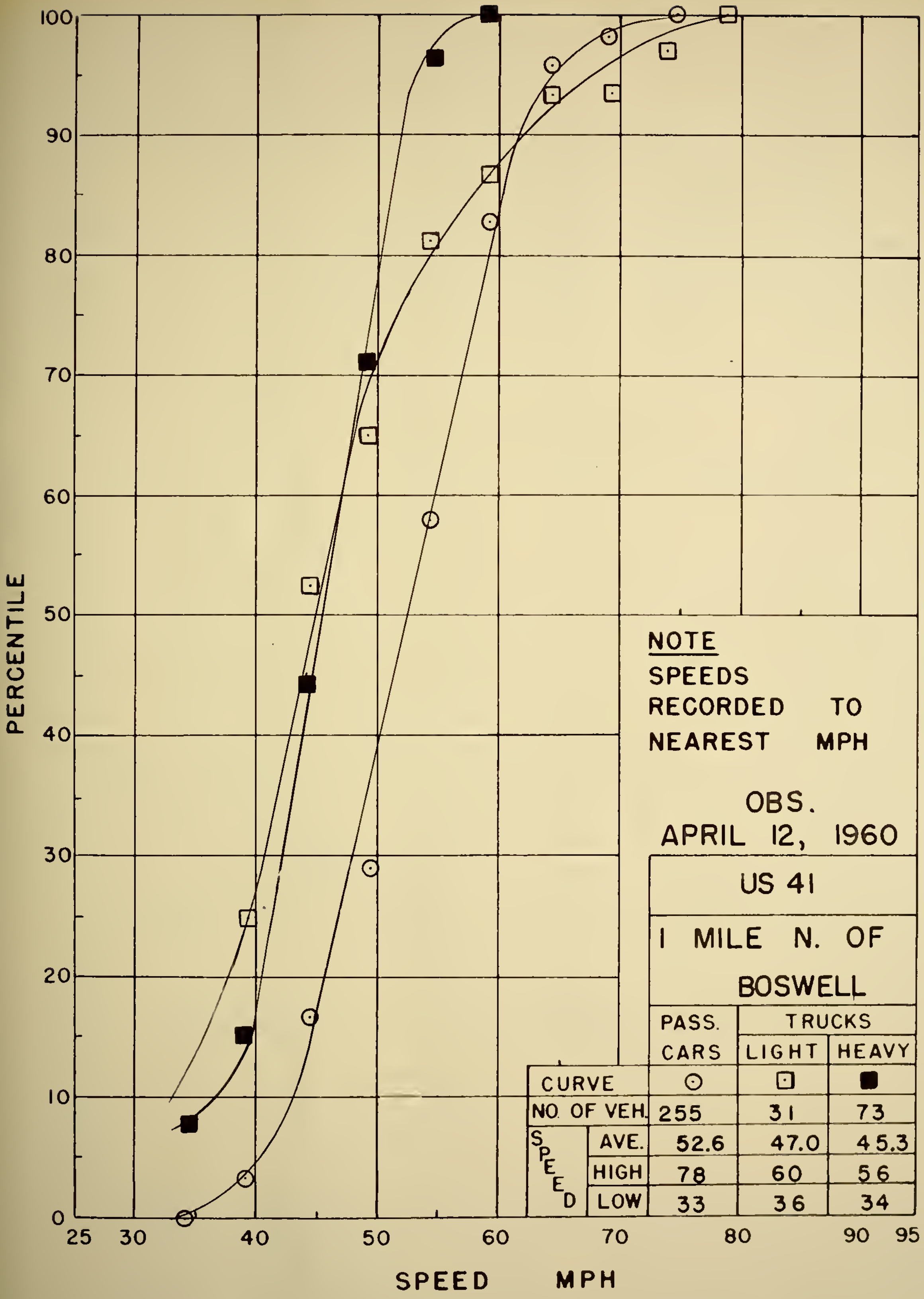
OBS
APRIL 1, 1960

SR 25

0.7 MILES W. OF AMERICUS

		PASS. CARS	TRUCKS LIGHT	TRUCKS HEAVY
CURVE		⊙		■
NO. OF VEH.		344	33	63
SPEED	AVE.	53.9	48	45.7
	HIGH	76	65	55
	LOW	36	37	40

FIGURE 5



NOTE
SPEEDS RECORDED TO NEAREST MPH

OBS.
APRIL 12, 1960

US 41

1 MILE N. OF BOSWELL

		PASS. CARS	TRUCKS LIGHT	TRUCKS HEAVY
CURVE		⊙	⊡	■
NO. OF VEH.		255	31	73
SPEED	AVE.	52.6	47.0	45.3
	HIGH	78	60	56
	LOW	33	36	34

FIGURE 6

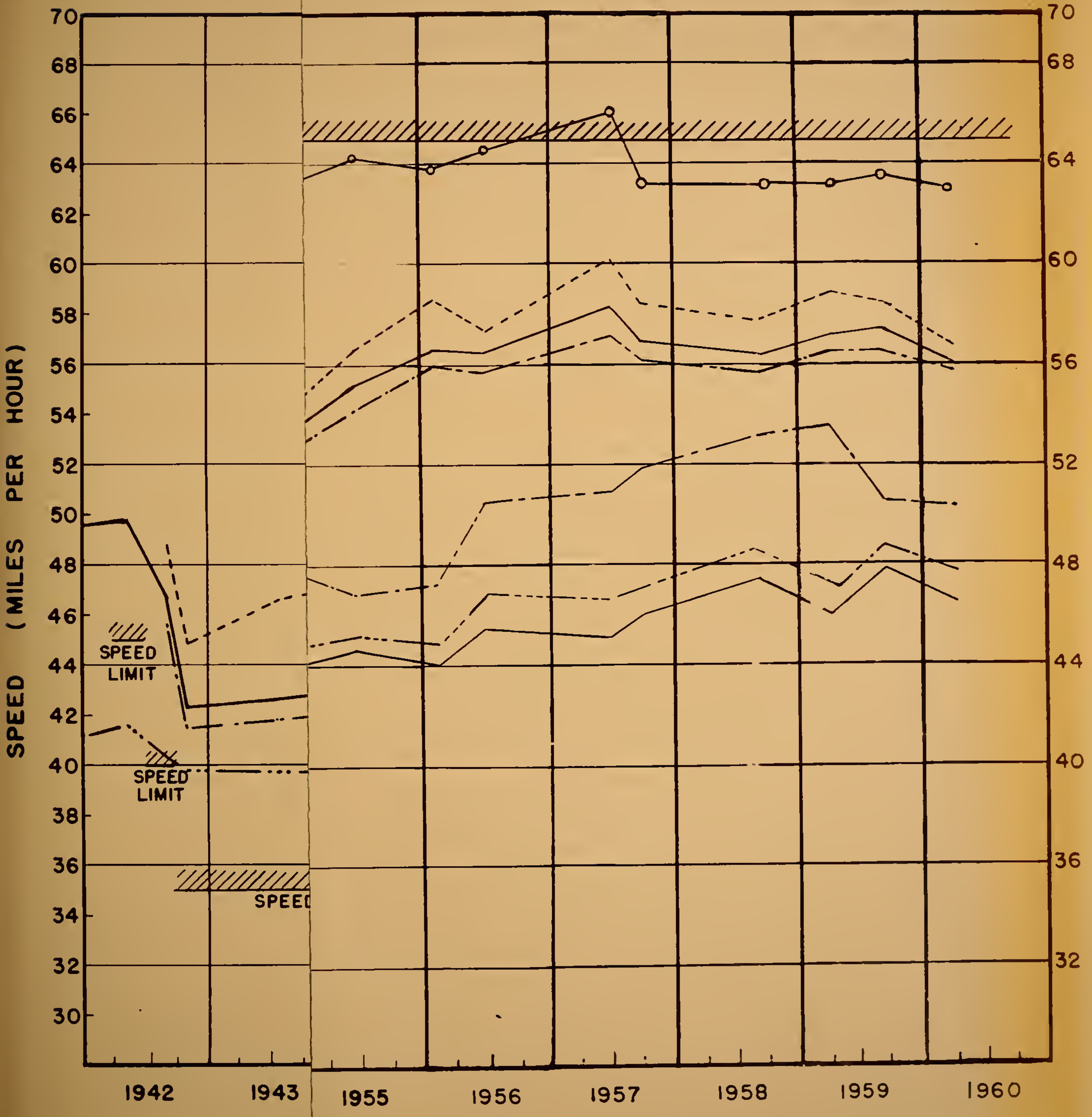
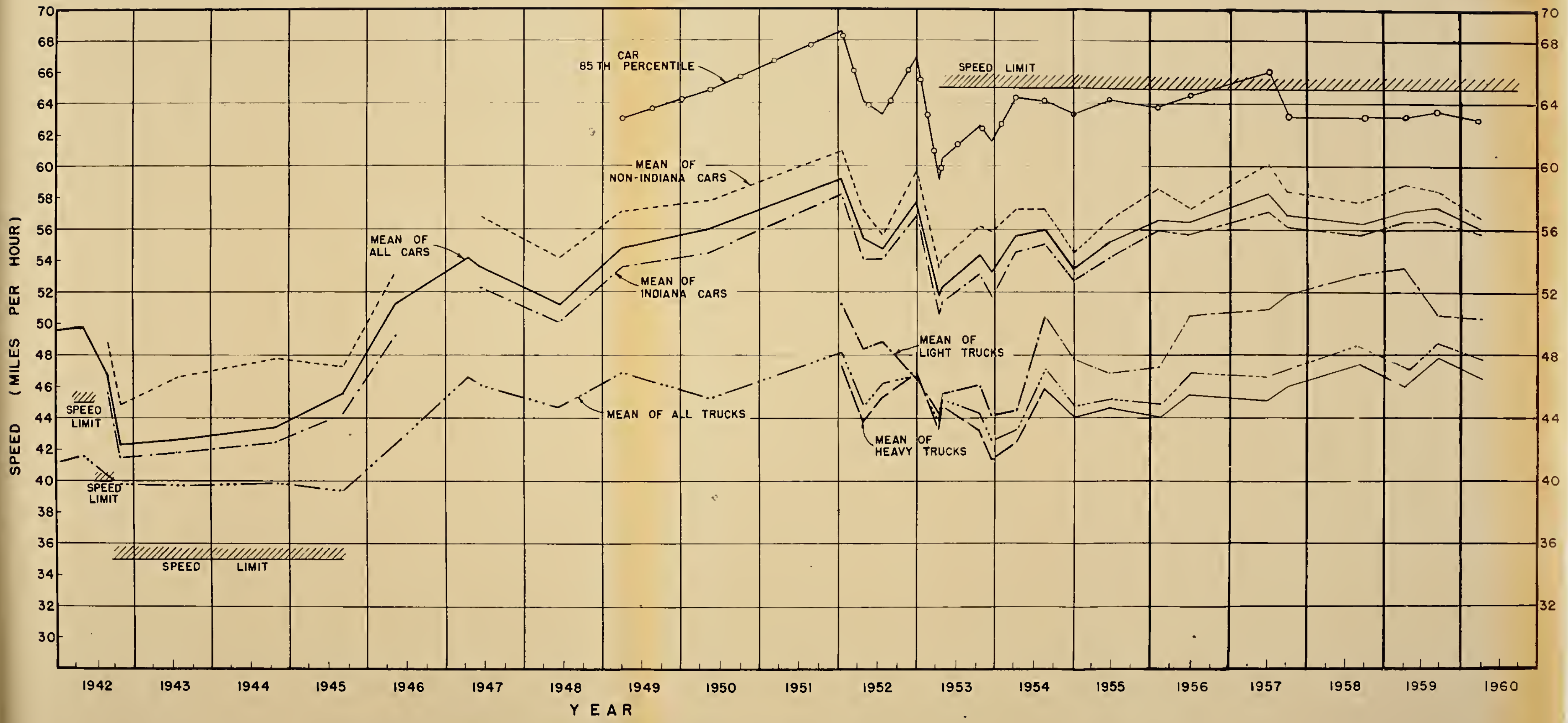
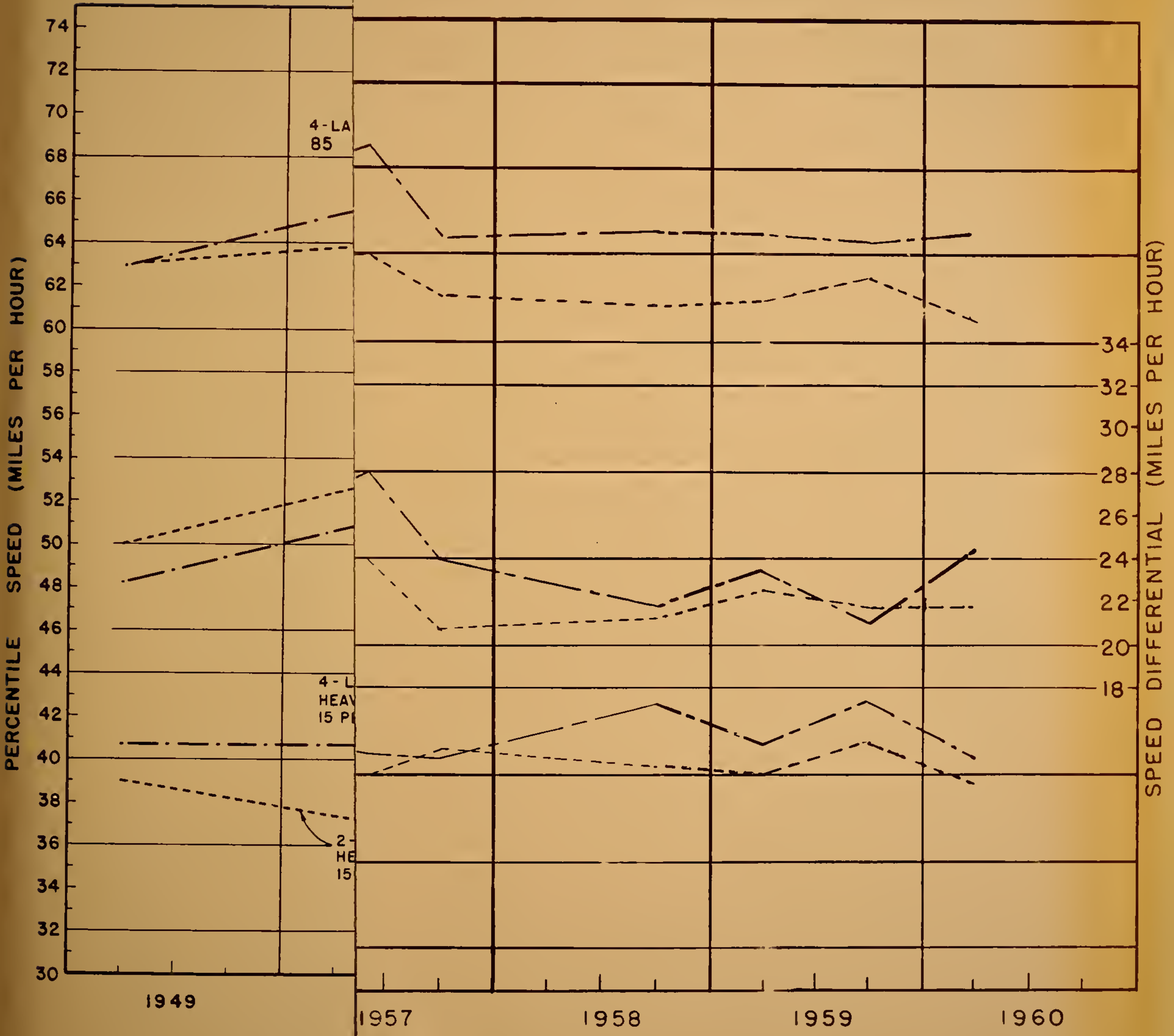

SPEED (MILES PER HOUR)
70
68
66
64
62
60
58
56
54
52
50
48
46
44
42
40
38
36
34
32
30
SPEED LIMIT
SPEED LIMIT
SPEED
1942
1943
1955
1956
1957
1958
1959
1960

FIG. 7



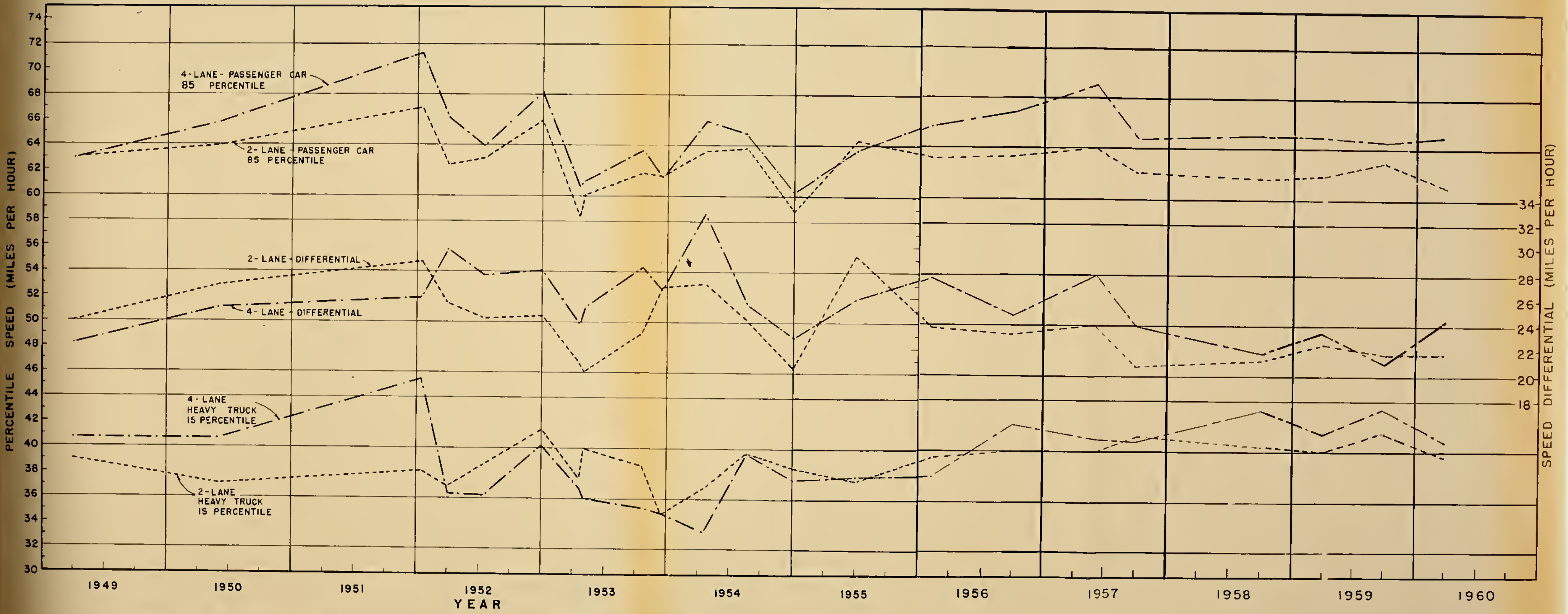
INDIANA RURAL SPEED TRENDS 1942—1960

FIG. 7


74
72
70
68
66
64
62
60
58
56
54
52
50
48
46
44
42
40
38
36
34
32
30
PERCENTILE SPEED (MILES PER HOUR)
4-LA
85
4-L
HEAV
15 P
2-
HE
15
1949
1957
1958
1959
1960
34
32
30
28
26
24
22
20
18
SPEED DIFFERENTIAL (MILES PER HOUR)

TRENDS

FIG. 8



TRENDS IN PERCENTILE SPEEDS AND SPEED DIFFERENTIAL 1949—1960

FIG. 8

C.E.
20.51
9NVe
960
c. 13

STRENGTH AND VOLUME CHANGE CHARACTERISTICS OF A BITUMINOUS MIXTURE UNDER TRIAXIAL TESTING

1119

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J. H. SCHAUB

Joint Highway Research Project

PURDUE UNIVERSITY
LAFAYETTE INDIANA

Final Report

STRENGTH AND VOLUME CHANGE CHARACTERISTICS OF A BITUMINOUS MIXTURE UNDER TRIAXIAL TESTING

TO: K. B. Woods, Director
Joint Highway Research Project
August 3, 1960

FROM: H. L. Michael, Assistant Director
Joint Highway Research Project
File: 2-4-16
Project: C-36-6P

Attached is a final report titled, "Strength and Volume Change Characteristics of a Bituminous Mixture under Triaxial Testing". It has been prepared by Mr. James H. Schaub, under the direction of Professor W. H. Goetz. Mr. Schaub also used this report as his dissertation for the Ph.D. degree.

The report is presented to the Board for the record.

Respectfully submitted,

Harold L Michael

Harold L. Michael, Secretary

HLM:kmc

Attachment

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F. F. Havey
G. A. Leonards
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J. E. Wilson
E. J. Yoder

Final Report

STRENGTH AND VOLUME CHANGE CHARACTERISTICS OF A BITUMINOUS MIXTURE UNDER TRIAXIAL TESTING

BY

James H. Schaub
Research Engineer

Joint Highway Research Project
File: 2-4-16
Project: C-36-6P

Purdue University
Lafayette, Indiana

August 3, 1960

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ABSTRACT

Schaub, James Hamilton. Ph.D., Purdue University, August, 1960. Strength and Volume Change Characteristics of a Bituminous Mixture Under Triaxial Testing. Major Professor: William H. Goetz.

A laboratory study was performed upon a bituminous mixture using the constant-lateral-pressure triaxial test and incremental static loading. The study was initiated to establish whether or not bituminous mixtures changed volume during testing and, if so, the magnitude of the changes and their relationship to strength. The effect of various conditions of drainage permitted during a test upon observed strength values was established.

The study was accomplished using a mixture composed of a constant aggregate gradation and varying quantities of a single bituminous cement. The mixture was compacted into specimens having uniform density and asphalt content throughout the specimen by the use of the kneading compactor.

Equipment was developed to permit measurement of the changes of volume of a specimen subjected to triaxial loading during the performance of a test. The equipment measured the change in volume of the triaxial cell liquid which reflected changes in the volume of the test specimen.

The stress-strain and volume change characteristics of sixty-five specimens were established. All tests were conducted at confining

pressures of 15, 30, or 60 psi and axial loads were applied through a lever system.

The results show conclusively, for the particular mixtures and procedures used, that bituminous mixture specimens change volume during the performance of a triaxial test. All specimens tested showed a slight decrease in volume followed by an increase in volume to failure. The volume increase was linear with axial strain from a strain corresponding to approximately minimum volume to failure. The results of the volume change studies indicated that dilatancy occurs in bituminous mixtures in much the same fashion as in dense granular soils.

Volume changes were analyzed in terms of the changes of percent voids and void ratios of the specimens. For the bituminous mixtures tested, a relationship between void ratio and percent voids of a specimen, at two percent strain and at the Proportional Limit, and the deviator stress at these points was established. This relationship was found to be linear between void ratio or percent voids (arithmetic scale) and deviator stress (logarithmic scale) and was dependent upon the initial void characteristics of the specimen. The relationship was apparently independent of confining pressure. To verify the unique character of the voids--deviator stress relationship, specimens were compacted to a given set of initial conditions and these conditions changed by water-saturation of the specimens prior to testing.

It was established, for the mixtures and procedures used, that the drainage conditions existing during a test had no measurable effect on the magnitude of observed shear strength values. Indirect evidence did indicate, however, that negative pore pressures developed during test-

ing. As the negative pore pressures did not affect the observed strength results, it was concluded that the effect of these pressures was so small as to be masked by the high strength values of the mixtures tested.

The effect of asphalt content and compaction pressure on the value of apparent cohesion and apparent angle of internal friction was investigated. The results clearly indicate that both of the apparent strength parameters were affected not only by asphalt content but by compaction pressure as well.

Results show that the aggregate density variation of the mixture used provided a fairly good measure of the relative values of observed compressive strengths while bulk density did not.

Data were gathered on the effect of asphalt content and compaction pressure upon the Modulus of Deformation of the mixtures studied. In general, it was established that the Modulus of Deformation was affected greatly by asphalt content at high compaction pressures but only slightly at low compaction pressures.

A limited study of the Deformation Ratio of the particular bituminous mixture used was accomplished. Deformation Ratio was defined as the ratio of lateral strain to axial strain and is analogous to Poisson's Ratio for elastic materials. The results of the study indicate that the Deformation Ratio varies from values of less than 0.5 to nearly 1.0 during a test. Further, it was shown that the apparent angle of shearing resistance was mobilized gradually with a continuing increase in Deformation Ratio to a point at which the maximum apparent angle of shearing resistance was developed. The maximum apparent angle

of shearing resistance established by the Deformation Ratio studies compared favorably with the same measure determined by the Mohr circle analysis of stress at failure.

INTRODUCTION

For approximately twenty-five years the attention of paving design engineers has been directed in part towards the use of a triaxial shear strength method of test to evaluate the properties of bituminous mixtures. Interest in this form of test, particularly as a means of evaluating fundamental strength properties of bituminous mixtures, has become increasingly great. At the present time there are several techniques in use that utilize the principles of triaxial shear testing for the design and strength evaluation of bituminous concrete mixtures.

The value of the triaxial shear test as a research tool is generally accepted; however, the utilization of this method of test for design purposes is limited. In comparison to other methods of test, the triaxial shear strength procedure attempts to evaluate the fundamental strength properties of a mixture under specified and known loading conditions. The stress system that acts upon a sample during the triaxial test approximates generally the system of stresses that are present during the loading of a mixture when it acts as a part of a pavement structure. Furthermore, the triaxial test makes use of a strength theory that agrees reasonably well with experimental results.

The problem of determining the strength characteristics of a material is of major interest in the study of all road-building materials. The field of soil mechanics has made the greatest use of triaxial shear testing for this purpose to the present time. This field of study has

developed a working explanation of the variation of measured shear strength with test procedure and with prior stress history for saturated soils. More recently, soil mechanics investigators have devoted their attention to the problem of establishing the shear strength of partially saturated soils. On the basis of the work done in soil mechanics, several general statements may be made regarding the shear strength of soils.

1. The shearing resistance of saturated, remolded cohesive soils is a function of the effective normal stress and the void ratio at failure.
2. The void content at failure of a partially saturated soil bears a unique relationship to the deviator stress at failure for a given set of initial conditions.
3. The previous two statements indicate that observed shear strength values are directly affected by the method of performance of the triaxial test and by the void content at failure.

An apparent similarity between a partially saturated soil, a three phase system composed of soil solids, water and air, and a bituminous mixture, composed of aggregate solids, bitumen, and air serves as a basis for an attempt to investigate the application of triaxial shear strength concepts developed in soil mechanics to the study of the strength of bituminous mixtures. Present practice in the triaxial testing of bituminous mixtures does not consider the effect of the method of performance of the test on observed shear strength values. Though air voids are of major interest in bituminous mixture design, little or no

attention has been devoted to the evaluation of the void content of bituminous mixtures at failure criteria.

It appeared desirable to establish whether or not the method of performance of a triaxial test on bituminous mixtures affects the observed shear strength of the mixture. In addition, a knowledge of the magnitude of change, if any, in the void content of a mixture during shear would aid in establishing a proper perspective of the role of air voids in the design of an adequate bituminous mixture for road use.

The study being reported represents an attempt to increase the fund of knowledge of the action of bituminous mixtures under triaxial loading. The specific purposes of the study were as follows:

1. To develop test equipment and procedures that would permit the study of the affect of the method of test on observed shear strength values and further permit the evaluation of the void content at any point during the progress of a test.
2. To develop a method of fabricating a sample suitable for triaxial testing that is reasonably uniform throughout with respect to density and asphalt content.
3. To establish whether the method of test procedure affects the observed shear strength of a bituminous mixture and the magnitude of this effect if any.
4. To establish whether there is a change in the volume of a bituminous mixture subjected to a

triaxial stress system and, if so, to establish the significant change in voids that is reflected by a change in observed shear strength.

5. To investigate the hypothesis that there is a unique relationship between the void content at failure and the shear strength of a bituminous mixture prepared to a given set of initial conditions.

The current status of knowledge of the triaxial shear strength of bituminous mixtures was established by a review of the available literature. The equipment developed for the study is presented. The tests used in an effort to satisfy the purposes of the study are reported and the results of these tests are discussed. Suggestions for research that would provide a logical continuation of this study are presented.

REVIEW OF LITERATURE

Numerous test methods have been developed and used with varying degrees of success for the design of bituminous mixtures. In all of these procedures, a measure of the stability of the mixture is obtained. Neppe (1) has classified the various stability tests as follows:

1. Compression
    a. Unconfined compression including immersion compression
    b. Constrained compression triaxial shear tests including both open and closed systems
2. Tensile and elongation tests
3. Bending tests
4. Shear tests
5. Extrusion tests
6. Indentation and penetration tests
7. Impact and toughness tests
8. Revolving drum brittleness tests
9. Punching shear tests
10. Vibration resistance tests
11. Abrasion tests
12. Load transmission tests including road machine and track service tests

By far the majority of stability tests performed on bituminous mixtures in the USA would be classified as compression tests by the above system.

In the continuing effort to develop an understanding of the mechanisms by which a bituminous mixture supports load and by which failure occurs, more and more attention has been directed to the triaxial compression test. In the opinion of numerous investigators the triaxial test is the only test procedure that offers the possibility of measuring stability in such a way that the stress system is subject to a rational analysis at all times during the test and also provides a stress system similar to that to which the mixture will be subjected in service. In illustration of this opinion, Monismith and Vallerga (2) state: "Stability is best defined as a load to cause a certain amount of deformation, said deformation depending upon expected field conditions, and that only a form of triaxial compression test will properly measure this property." Smith (3) has written: "In this test (triaxial) method the stresses acting upon the specimen during testing closely approach the system of stresses existing in a flexible pavement or pavement foundation when they are supporting a load. Triaxial shear theories are in close agreement with actual experiment." In reference to the stability of one-size aggregate mixtures, Oppenlander (4) states: "Excluding empirical test methods, the triaxial compression test is the only suitable method for the fundamental evaluation of the true stability of this bituminous mixture having a one-size aggregate."

The purposes of this study are basically an effort to further advance the fund of knowledge concerning the action of bituminous mixtures

under triaxial loading in the belief that the foregoing statements correctly present the value of this type of test.

The literature review for the study required the perusal of numerous articles covering a wide range of topics relating in some way to the purposes of the study. For simplicity's sake, the literature review is presented in three parts:

1. Triaxial Testing of Soils,
2. Triaxial Testing of Bituminous Mixtures,
3. Relationship of Air Voids to Mixture Design.

Triaxial Testing of Soils

Expressions for the shearing resistance of earth masses have been based historically upon the hypothesis of Coulomb (5) which was first advanced in 1776. This hypothesis may be summarized algebraically in modern terminology as follows:

$$s = c + \sigma \tan \phi \qquad \text{Equation 1}$$

in which s is the shearing resistance of the material; c is the unit cohesion; σ is the value of the normal stress on any portion of a plane in the material; and ϕ denotes the angle of internal friction of the material. This expression of shearing resistance considers both c and ϕ to be constant for any given material. The Coulomb equation and the accompanying concept of constant shear-strength parameters have influenced the expression of shearing resistance of soils since its elucidation.

Beginning in 1925, the work of Dr. Karl Terzaghi called the attention of engineers to the influence of pore water pressure and the effect

of this pressure on the results of shear tests. In 1936, Terzaghi (6) pointed out that the σ term of the Coulomb equation must be considered as the effective normal stress (total normal stress less pore water pressure) on the plane of failure. Further, the Terzaghi Theory of Consolidation (7) implies that the magnitude of pore water pressure developed as a result of loading is directly affected by the amount and rate of drainage permitted during the test. This implication was borne out by the subsequent investigations of Jurgenson (8) and others at the Massachusetts Institute of Technology.

The concepts of the shear strength of soils were drastically revised as a result of the independent investigations of Hvorslev (9) and Rendulic (10). The conclusions of these investigations include the following:

1. During shear a cohesive soil will tend to expand or to decrease in volume depending upon the state of consolidation of the specimen.
2. The pore-water pressure will be equal to atmospheric pressure at all times if the rate of load application is sufficiently slow so that the void ratio of the soil may become fully adjusted to the prevailing stress conditions at any given time.
3. Under conditions of stress application that is so rapid that full adjustment of the void ratio cannot be accomplished, the pore pressures will be in excess of atmospheric pressure if the sample tends to decrease in volume. If the sample tends to increase in volume under similar

stress application conditions, the pore pressure will be lower than atmospheric pressure.

4. The shearing resistance of saturated, remolded cohesive soils is a function of the effective normal stress and the void ratio or water content at the moment of failure.
5. The true angle of internal friction of such soils is a constant; however, the cohesion of the soil is a function of the void ratio at failure.

On the basis of the Hvorslev investigations, the Coulomb equation may be written in a more fundamental form for saturated soils:

$$s = c_e + (\sigma - u) \tan \phi_e \qquad \text{Equation 2}$$

where c_e is the true cohesion of the material, ϕ_e is the true angle of internal friction and is essentially a constant, and u is the value of the pore-water pressure or neutral stress.

The validity of the Hvorslev concepts has been verified by numerous investigations in Europe (11)(12)(13). Skempton and Bishop (11) state unequivocally, "In the authors' opinion it (the Coulomb-Hvorslev equation) must form the basis of any fundamental considerations of shear strength."

In the practical determination of the shear strength of a soil the application of the Hvorslev concepts of true cohesion and true angle of internal friction involves the measurement of pore pressures developed during the test. In the interest of simplicity and reliability the use of

test values of c and ϕ determined under test conditions similar to those expected in the full-scale structure are often used in lieu of the more correct c_e and ϕ_e values. Rutledge (14) credits A. Casagrande with the proposal for the following three basic types of tests which limit the conditions of pore pressure and intergranular stress that is acting and establish stress systems similar to expected field conditions:

> "a. Quick tests (Q). In these tests no drainage of pore water from the specimen is permitted. Neglecting any possible effects of axial deformation on a tendency toward volume change in the soil, any applied stress creates an equal pressure in the pore water. The simple unconfined compression tests (U) properly performed on saturated clays are tests of this type.
>
> "b. Consolidated-quick (Q_c). In these tests complete consolidation, under an applied pressure that is equal in all directions or hydrostatic, is permitted. The axial stress is increased to failure without further drainage of pore water. Again neglecting effects of axial deformation on tendency toward volume change, the pore pressure is equal to the axial deviator stress, in other words the axial stress that is added to the initial conditions of externally applied hydrostatic pressure.
>
> "c. Slow tests (S). In these tests complete drainage of pore water is permitted under both the applied hydrostatic pressure and the subsequent increase in axial stress. In a perfect slow test the pore water pressure is equal to zero throughout the test."

All of the concepts stated in the preceding discussion are applicable to saturated clays. The test procedures may be applied to either direct shear tests or to triaxial compression tests. For reasons of better test control and more rational test conditions the triaxial compression test is preferred for the shear testing of soils. Rutledge (14) has provided the following complete definition of triaxial compression:

> "Triaxial Compression designates a condition of test in which stresses of known magnitudes in three rectangular directions are applied to a specimen that is free to change shape in three directions. The stresses may be equal in three directions, equal in two directions, or unequal in all three rectangular directions. The only required condition is that the stresses cause a shortening of the specimen in the direction of its axis. The common conditions of stress in triaxial compression are equal stresses in three coordinate directions, called triaxial consolidation because the stresses cause only volume decrease in the specimen, and equal stresses in two lateral directions with a greater stress in the axial direction. The latter case is the stress condition commonly known as triaxial compression and, in general, it causes both volume changes and shear deformations in the specimen."

In simple terms the triaxial compression test may be considered as a compression test in which the specimen is supported throughout its length by equal stresses acting in all lateral directions.

In summary, the knowledge regarding the shear strength of saturated clays is well expressed by Rutledge (14): "The maximum principal stress difference ($\sigma_1 - \sigma_3$), usually called the compressive strength of clay, depends only on water content at maximum stress, provided the test specimens have not been preconsolidated under pressures greater than the test maximum stress and then allowed to rebound to equilibrium under lower pressures." It should be noted that a similar and equally valid statement may be made using void ratio as a reference rather than water content as these characteristics are proportional to each other for a saturated soil.

With regard to the strength of cohesionless soils, Rutledge (14) reports that the strength is dependent upon the angle of internal friction of the material which is a function of soil type and the density of the specimen at the time of test but is independent of the minor principal

stress and method of test. The effects of drainage during testing of cohesionless soils are clearly established by Casagrande (15) who called attention to the fact that cohesionless soils tend to change volume during shear. Whether or not the volume change is expansion or a reduction in volume is dependent upon the density of the specimen. Loose specimens tend to be reduced in volume while dense specimens tend to expand. If such a material is saturated, the tendency to change volume effects pore water pressures and consequently the strength of the material. The void ratio of a soil at which neither expansion or reduction in volume occurs during shear was given the name 'critical void ratio' by Casagrande.

The state of knowledge concerning the shear strength of partially saturated soils is not so well established as that for saturated materials. Rutledge (14) reports:

> "Four major variables effect the strength of a soil in this group. These are: the minor principal stress, the dry soil density, the water content, and the percent of voids filled with water, usually called the degree of saturation. The last two variables are not independent. ...In addition, for compacted soils the method of and conditions during compaction seem to have independent effects on test strength."

A recent study of the compression characteristics of partially saturated soils by Yoshima (16) makes the following statement: "In contrast to the saturated system, an important feature of the partially saturated system is the fact that the compressibility of air allows immediate compression of the spring (soil skeleton) without the need of displacing fluids out of the system." A similar result was reported in a comprehensive study of the shear strength of partially saturated soils by Leonards (17) as follows: "...during the application of

shearing stresses on partially saturated clays, changes in void ratio occur that are not accompanied by corresponding changes in water content. The nature of these changes in void ratio depends on the initial conditions, the confining pressures, the preconsolidation pressure, and the quantity of water added to the soil from an external source."

The change in volume of partially saturated soils without change in water content during triaxial shear has been noted by Holtz (18), Taylor (19), Wagner (20) and others. Holtz explains this action as follows: "The void spaces of a soil mass are filled with a partially compressible fluid which consists of a mixture of gas and water, the compressibility of the mixture being dependent upon the relative proportions of gas and water. Loading a specimen in shear produces volume changes which must be accompanied by an extrusion of the pore fluid from the soil mass or by compression of the fluid."

Wilson (21) points out that the air in the voids of a partially saturated soil acts in the same fashion as water in reducing the effective stress developed under load. "The distribution between effective and neutral stress upon loading depends not only on the original degree of compaction achieved, but on changes in void ratio and degrees of saturation that may develop in the course of time. Therefore, the strength properties of a compacted mass will differ at different elapsed times after compaction." These comments would indicate that it is possible for the strength of a compacted material to vary with time even though no water content changes occur.

With the consideration of the variables involved in the shear strength of a partially saturated soil it is apparent that no simple

relationship can predict the strength characteristics of such materials. Rutledge (14) noted that the following three test characteristics exist for partially saturated soils:

"a. In Q-tests the principal stress difference, or compressive strength, increases with lateral pressure.

"b. Very little difference in strength is observed in the results of Q- and Q_c-tests.

"c. The relation between compressive strength and water content at the end of test varies with the minor principal stress."

Leonards (17) concludes that for a given set of initial conditions,

> "...the pattern of strength variation for partially saturated, compacted clays can be represented by a relationship between the void ratio at failure (plotted to an arithmetic scale) and the maximum value of the principal stress difference, or compressive strength (plotted to a logarithmic scale). For a specified set of initial conditions this relationship is independent of the confining pressure, the amount of drainage permitted, the water content, and the degree of saturation. It is indicated that the relationship is unique even if the degree of saturation (or water content) is increased by the addition of water from an external source, and there is a possibility that this uniqueness will exist even after the soil has been preconsolidated and then allowed to rebound fully at a lower confining pressure."

The analysis of the results of a triaxial compression test commonly is accomplished by the application of the Mohr Theory of Failure (22). In general, the Mohr theory hypothesizes that the limiting shear stress depends on the normal stress acting on the plane of failure and that failure depends on the maximum magnitude of the difference in principal stresses acting assuming that the intermediate principal stress does not effect the shear strength. Graphically, the Mohr theory is expressed as a locus of points (called the envelope of failure) whose coordinates are the values of normal and tangential components of stress

on the plane of failure resulting from paired values of major and minor principal stresses occurring at a point of incipient failure. A circle constructed with the maximum difference in principal stresses as a diameter is tangent to the envelope of failure and represents the stress components on all planes through a point in the stressed mass.

Cohesionless soils generally exhibit a straight line envelope of failure though a curvature of the failure envelope may be observed at high confining pressures. Normally consolidated, saturated cohesive soils exhibit a straight line envelope of failure while pre-consolidated, saturated cohesive soils display a curved envelope for confining pressures less than the pre-consolidation pressure. The failure envelope is a graphical representation of the Coulomb equation. The slope of the envelope defines the angle of internal friction while the intercept of the envelope with the vertical axis establishes the cohesion value. The parameters established by this analysis are apparent values if total stresses are used to determine the envelope of failure. The apparent parameters are a function of the drainage conditions permitted during the test or of the effective stresses acting during shear. The true angle of internal friction and true cohesion as used in the Hvorslev modification of the Coulomb equation (see Equation 2) may be defined using samples of a material in both the normally consolidated and pre-consolidated states that have the same void ratio at failure.

In the case of partially saturated cohesive soils, the Mohr envelope frequently shows a curvature that is concave downward for an undrained or Q-test. Bishop and Henkel (23) explain this curvature as follows: "The deviator stress at failure is found to increase with cell pressure. This increase becomes progressively smaller as the air in

voids is compressed and passes into solution, and ceases when the stresses are large enough to cause full saturation." For the consolidated-undrained or Q_c-test, the curved envelope will develop throughout the lateral stress range below that required to compress all the air voids or to develop complete saturation. The curved envelope of failure is not associated with the drained or S-test due to the complete drainage that is permitted under all applied stresses. In all cases an envelope of failure prepared using effective stresses may be constructed that will permit the definition of the true shear strength parameters.

Triaxial Testing of Bituminous Mixtures

Partially saturated soils and bituminous mixtures are analogous three-phase systems. The former consists of a solid phase of soil particles, a liquid phase composed of water, and a gaseous phase that is the air in the unfilled voids. Bituminous mixtures are composed of a solid phase which consists again of solid particles or the aggregate, a liquid phase formed of the bitumen, and a gaseous phase consisting of the air in the air voids of the mixture. Because of the striking analogy between these two widely used construction materials, it seems to be a logical step to attempt to analyze the shearing resistance characteristics of bituminous mixtures in light of the available knowledge and methods established in soil mechanics.

The desirability of using the triaxial test for the analysis of the strength characteristics of road materials was voiced by Palmer (24) in 1938: "It was the concensus of this meeting (Eighteenth Annual Meeting, Highway Research Board) that the triaxial compression or stabilometer device is the most useful shearing test method and despite all obstacles it is proposed to obtain and use complete stress-deformation diagrams in connection with highway problems." The asphalt paving

industry has been aware of the potential offered by the triaxial method of test for many years. As early as 1934, Stanton and Hveem (25) made mention of the early forms of the stabilometer and a test procedure that involved supporting a specimen laterally while deforming it by the application of an axial load.

Beyond the logical extension of the knowledge of the shear strength of soils to the similar bituminous concrete systems, the triaxial method of test offers other advantages for the study of the strength of bituminous mixtures. Smith (3), in discussing the triaxial test method has stated, "In this method the stresses acting upon the specimen during testing closely approach the system of stresses existing in a flexible pavement or pavement foundation when they are supporting a load. Triaxial shear theories are in close agreement with actual experiment." Endersby (26) pointed out:

> "A body of material in a pavement is subjected to a combination of vertical load and lateral restraint. ...experimental means of measuring the effect of this combination are called for. In addition, an analysis of the respective functions of binder and aggregate require a method which shows the cohesion and friction as separate forces. All these conditions are fulfilled by the triaxial test when combined with the Mohr diagram type of representation of results."

A further advantage of the triaxial test for evaluating the shearing resistance of bituminous mixtures is the ease with which the procedure may be adapted to the particular needs of research. Goetz (27) has summarized this advantage as follows: "As a result of this rather extensive investigation, we favor the use of the triaxial test method as a research tool for the evaluation of mix variables, and as a basis on which to obtain data to extend present knowledge of bituminous

pavement design." The advantages of the triaxial test for research studies were reiterated by Goetz and Schaub (28) by emphasizing the versatility of the triaxial method of test in the investigation of the numerous variables in bituminous mixture design.

Triaxial testing methods for bituminous mixtures have developed along two similar but distinctly different lines. The two general procedures often are called the 'closed' system and the 'open' system. The open system is similar in form to the tests used for soils as previously discussed. A lateral pressure is applied to the specimen and held constant while an axial load is applied to cause failure of the specimen. In contrast, the closed system permits the lateral pressure to vary as a result of the tendency of the sample to deform under an applied load. It is likely that Hogentogler (29) first introduced these terms in defining various methode of triaxial test during a summary of triaxial shear tests presented in 1939. Hogentogler's classification states: "Methods employed included a 'closed system' which prevents volume change of samples, and an 'open system' which permits swell or consolidation during test. An impervious casement which prevents entrance or escape of air and water encloses samples in the closed system and placing them between porous stones provides for the entrance or egress of air and water in the open system." The primary advantage claimed for the closed system is that a value of c and Ø for a mixture may be established from a test performed on only one sample whereas a minimum of two and preferably three or more samples, tested at different lateral pressures, are required for this purpose when using the closed system. For simplicity's sake it seems desirable to refer to the

open-system test as a constant-lateral-pressure test and to the closed system as a variable-lateral-pressure procedure.

A second major divergence in the test procedures presently in use for testing bituminous mixtures is the relationship between the height (H) and the diameter (D) of the specimen tested. One group of test procedures, notably those represented by the Stabilometer method, utilizes a sample in which the height is less than the diameter of the specimen. The adherents of this technique argue that such a sample is similar to the prototype existing in the roadway and, therefore, test results may be considered as representative of pavement performance. Other triaxial tests are performed using a sample in which the height is equal to or greater than the diameter. In such samples, shear planes are free to develop within the sample and a theoretical analysis is valid. A sample with a height that is small relative to its diameter will have confining stresses induced at the ends of the specimen by the loading heads. Rutledge (14) notes that samples with an H/D ratio of two to three are satisfactory for soils. H/D ratios greater than that noted may not be representative due to the possible introduction of column action in the sample. The effects of a low H/D ratio have been discussed by Pheiffer (30), Smith (3) and others.

Unless otherwise noted all references to triaxial testing in this discussion refer to samples with an H/D value of approximately two and to an open-system or constant-lateral-pressure method of test.

Effect of the Binder

The analysis of the results of triaxial tests of bituminous mixtures is complicated by the nature of the binder and by the relationship

between the binder and the aggregate component. Lee and Markwick (31) have stated: "From the point of view of their mechanical properties bituminous materials exhibit the characteristics of (1) plastic materials, which flow under sustained load, (2) elastic materials, which show pronounced hardness and elasticity and characteristics of solids, (3) granular materials characterized by (a) void content and porosity, (b) consolidation, (c) dilatancy when subjected to deformation."

The observed shear strength of a bituminous mixture established by triaxial testing is affected by the viscosity of the binder. Nijboer (32) considers this phenomena as follows:

> "When using this test to the fullest extent the resistance to shear of the material under conditions prevailing in road work can be described by the three following physical magnitudes: (1) a frictional resistance, due to friction between the aggregate particles, which exert a pressure on each other (grain pressures) and is proportional to this pressure. The angle of internal friction is used as its characteristic in accordance with soil mechanics. (2) An internal resistance, a resistance the material is able to develop in the absence of frictional resistance. (3) A viscous resistance (in poises), developing only when the material is sheared off at a certain rate. This resistance is due to the presence in the material of the viscous asphalt cement. Attention should be drawn to the viscous resistance that even at low rates of deformation can acquire high values."

With the effect of the viscosity of the binder entering the concepts of shear strength of bituminous mixtures so profoundly, it is obvious that any variable influencing viscosity will also effect shear-strength measurements. The effect of rate of load application has received a great deal of study and comment. Goetz, McLaughlin, and Wood (33) have shown that the maximum compressive strength increases with increasing rates of load application. Goetz (27) has found that

cohesion parameter is drastically increased with an increase in testing speed but that the friction parameter is essentially constant over a wide range of testing speeds. Smith (3) suggests that a test procedure that eliminates the viscosity effects resulting from the rate of load application is desirable. "Application of static loads in increments is essential in order to eliminate the viscous resistance of the asphalt binder which is directly proportional to the rate of strain. It has been well established that static loads represent the severest stress conditions imposed upon bituminous pavement because under continuous stress conditions the viscous resistance of the asphalt or similar substances is essentially zero." Endersby (26) reached the same conclusion. Stevens (34) goes further and states: "The stability component resulting from the viscosity of the asphalt in a mix cannot be relied on as it becomes small or negligible under high temperatures and slow moving traffic or static load conditions, such as are experienced on city streets and airfield taxi-ways." The complexity of the problem of rate of load application and stability is summed very well by Stevens (34):

> "How much of the viscous resistance of the asphalt should be relied upon in judging the long time stability properties of an asphalt mix? It is true that it contributes to the stability of pavements under moving loads, and it is also true that a majority of the loads are moving. However, can one count on loads to be moving, or should one design for the worst condition, the static load, and use the viscous resistance of the asphalt as an added safety factor?"

In a similar fashion, variations in the temperature of the specimen at the time of testing affect the viscosity of the binder and so influence the observed shear strength of the mixture. Smith (3) has

written: "...the resistance of an asphaltic binder to imposed strains is dependent upon the consistency (viscosity) of the binder which in turn is affected markedly by change in temperature." The effects of test temperature on maximum compressive stress have been indicated by Goetz, McLaughlin, and Wood (33). In general, the maximum compressive stress observed is greater for lower temperatures for any given rate of deformation. Neppe (35) has found: "The mechanical stability of a bituminous mixture at any temperature, is approximately a direct function of log viscosity of the contained binder at that temperature and is independent of the source, nature, and properties of the latter constituent." Wood and Goetz (36) and McLaughlin (37) studied the relationships existing between compressive strength, temperature and rate of deformation and have established that for the mixtures and conditions studied the relationship could be expressed mathematically. In the opinion of Endersby (26) a test temperature of 77°F is a reasonable testing temperature as this value probably represents a good year round average and serves to bring out the role of the binder more clearly than a higher test temperature.

Influence of Sample Preparation

An additional factor influencing the observed strength values of a bituminous mixture is the method by which the sample is formed. Endersby and Vallerga (38) state: "The method of forming laboratory test specimens, even apart from corresponding bulk density, has a profound effect on test results, regardless of the test method or its validity." Basically all of the schemes in use for fabricating samples fall into one of three categories: (1) dynamic compaction as illustrated by the

compaction procedure utilized in the Marshall test (39); (2) static compaction which is one of the established procedures for preparing samples for use with the Smith Triaxial Cell procedure (40); and (3) kneading compaction which includes the procedure advocated by the adherents of the Stabilometer method of design (41), as well as the gyratory shear procedure (42) which is presently receiving extensive study.

The question of which compaction procedure is best suited for preparing samples for testing has received continuing study and comment. As the many design procedures in use are essentially empirical, the compaction procedure utilized should be the one that satisfies the correlation that forms the basis for the development of the particular design procedure. There is evidence, however, that different compaction procedures produce different particle orientations. It would seem reasonable to strive for a compaction procedure that produces a particle orientation similar to that obtained by compaction under field conditions. Extensive studies by a group of materials engineers on the Pacific Coast, generally known as the Triaxial Institute, investigated the kneading compaction procedure. Endersby (43) presents as one of the results of these studies: "...only kneading or rolling methods are likely to reproduce road conditions." Investigations by Vallerga (44), Hveem and Vallerga (45), Ortolani and Sandberg (42), and the experience of several agencies with the kneading compactors tend to substantiate this conclusion. The experience gained in preliminary studies of compaction procedures for this investigation led to the conclusion that samples of the desired height and having a reasonably uniform density

could be best obtained by using a kneading compactor. (See "Materials and Procedures".)

Criteria for Strength Evaluation

Still another basic factor which must be considered in a study of the characteristics of bituminous mixtures is the criterion to be used to judge the strength or 'stability' of the mixture. The plastic characteristics of bituminous mixtures permit such materials to accept large loads without shear failures but with high deformations. For design purposes, the stability of the mixture must be chosen with a concept of some limiting value of deformation in mind. Stanton and Hveem (25) define stability as "that property of bituminous pavements which tends to resist plastic deformation". Monismith and Vallerga (2) consider: "Stability is best defined as a load to cause a certain amount of deformation, said deformation depending upon expected field conditions...." For the purposes of this study, stability is considered as the difference between the major and minor principal stresses causing a given deformation of a specimen in a triaxial test. Compressive strength is considered as the maximum difference between the principal stresses acting at failure.

For any given compaction process and a fixed aggregate blend there is a variation in the density of the compacted specimen with variation in asphalt content. In general, as asphalt content increases there is an increase in density to a maximum value followed by a decrease in density with further increases in asphalt content. Lee and Markwick (31) explain the development of maximum density with increasing asphalt contents as follows:

> "The volume occupied by a consolidated mixture composed of a given quantity of aggregate with varying amounts of binder will be determined by the binder content, other factors remaining constant. Small percentages of binder act as a lubricant and assist the aggregate to attain its closest packing. The binder is present as a thin film surrounding each particle of aggregate, and additional binder causes an increase in thickness of these films, thereby reducing the frictional resistance to compaction. Hence up to a certain point, there is diminution in the total volume as the binder content is increased. After this point has been reached, further additions of binder, by increasing the film thickness, will cause an increase in volume."

With the realization that the cohesion component of the shear strength of bituminous mixtures is affected by asphalt content, the relation between the optimum asphalt content and density for a given set of materials to the maximum shear strength of the mixture becomes of critical importance. Neppe (1) and Endersby (26) point out that the presence of the binder complicates the determination of the stability of the mixture. Increased quantities of bitumen increase the cohesion of the mixture but simultaneously it acts as a lubricant which tends to reduce stability. Mixture design methods often indicate that maximum stability of a mixture is achieved with the combination of aggregate and bitumen that produces maximum density. This observation, in general, does not hold true for triaxial test evaluations of shear strength, where it is usually observed that maximum stability occurs for asphalt contents less than those required for maximum density. Goetz and Chen (46), for example, note that their test results show, "...maximum stability at any confining pressure does not occur at the asphalt content which yields maximum density. The asphalt content at maximum stability is less than the asphalt content at maximum density."

Analysis of Strength Test Data

A graphical representation of the results of an open-system triaxial test series is normal procedure for the analysis of the shear strength of bituminous mixtures. Utilizing the Mohr theory, the shear strength of the mixture is presented as a cohesion and a friction component. An extensive search of the literature on triaxial testing of bituminous mixtures has failed to reveal any detailed discussion of the effects of drainage conditions during the test on shear strength. If drainage conditions during the test affect observed shear strength values, then the triaxial test data available must be considered as c and ∅ values apropos to the particular test conditions under which they were determined. This consideration makes it impossible to compare results from different agencies and furthermore, it limits the determination of the fundamental shear resistance characteristics of bituminous mixtures.

There is direct and implied test evidence to indicate that there is a form of drainage from a bituminous-mixture specimen under load. The drainage most generally occurs from the air phase, but the asphalt also may be forced from the specimen under load. Campbell (47) investigated the one-dimensional compressibility of bituminous mixtures. His results indicate that under an applied load there is relatively sudden compression of the mixture followed by a lesser and slower reduction in the height of the sample with time. The former compression was thought to be due to compression and drainage of air from the sample while the latter was considered to be due to movement of asphalt. Campbell noted further: "It was observed at the completion of each test

that asphalt had actually been expelled from the samples and forced into the faces of the porous stones." McCarty (48) points out that the application of the Coulomb equation to bituminous mixtures must be revised in light of the Hvorslev modifications that have been accepted by soils engineers (see "Triaxial Testing of Soils"). Numerous other authors refer to changes in density of specimens during testing which must be considered as an implication of drainage either from the specimen or into it. Nijboer (49), for example reports: "If it is realized that in the triaxial test all principal stresses on the test piece increase, involving a reduction in volume of the specimen, it must be inferred that the void content of the aggregate will be reduced during testing."

If the hypothesis of the possibility of drainage of some type from a sample of a bituminous mixture subjected to triaxial testing is accepted, it would seem desirable to establish the effects of test drainage on observed shear strength and stability values.

Using the existing test procedures (no particular regard to drainage) and the Mohr circle analysis, the phenomena of the curved envelope of failure has been observed frequently. The occurrence of the curved envelope has caused extensive concern among paving engineers and no explanation of the cause is accepted universally. McLeod (50) candidly states: "Not enough appears to be known about the triaxial testing of bituminous mixtures to establish why straight line Mohr envelopes are obtained for some and curved Mohr envelopes for others." The wide divergence of authoritative opinion on the cause of curved envelopes of failure is well illustrated by the following quotation from McLeod (51).

> "Professor Vallerga states that 'the curvature of Mohr's envelope of failure and the apparent intercept on the ordinate for these materials are merely effects produced by particle size relative to specimen size, size of specimen, kind of loading, limitations of test equipment and instrumentation, and finally improper interpretation of test data'. Mr. Endersby believes that 'the curvature represents the structural effect pure and simple, and the extent of it depends upon the dimensions of your sample'. According to Mr. Carpenter, 'we get curvature when there is a change in density of the specimen during test', while Dr. Mack states, 'the hardening effect can be also demonstrated by a curved envelope to the stress circles.' ...It is quite apparent, therefore, that considerable difference of opinion exists concerning the cause of the curvature of the Mohr envelope."

Nijboer (49) notes, "With respect to Mohr's diagrams, it should be stated that the mixtures with a low bitumen content show the normal picture, but for mixtures containing more bitumen and having a void content below 2-3% by volume the enveloping curve of the circles is no longer a straight line, but runs concavely towards the σ-axis (horizontal axis)."

McCarty (48) believes, "The deviation of the Mohr envelope from this straight line relation may be interpreted as the effect of a change in cohesion, due to compaction of the specimen, or as a consequence of changes in structural properties of the material due to internal rearrangement of aggregate particles in the mix, etc., which is a necessary accompaniment of the changing consolidation." Eatman (52) reiterates McCarty's belief.

Considering the weight of evidence that bituminous mixture specimens change in density during a triaxial test and recalling the explanation of the curved envelope of failure that has been accepted for partially saturated soils, it is likely that a similar explanation is

reasonable for bituminous mixtures. If this be the case, the need for establishing the effect of test drainage conditions on observed shear strength and accurate measurements of the change in volume of the specimen under all loads applied during the triaxial test becomes more critical. It is this line of thought that led to the research reported herein.

Relationship of Air Voids to Mixture Design

Any study that contemplates measuring the change in volume of a bituminous mixture specimen under load must consider the question of the voids in the mixture. If it is assumed that the aggregate particles and the bitumen are incompressible, then a decrease in sample volume may be accomplished only by a decrease in air-void volume.

The majority of established mixture design procedures presently in use specify or suggest limits to the amount of voids in the compacted specimen. The concepts reflected in the establishment of void limitations are expressed well in Highway Research Board Bulletin No. 105 (53):

> "In designing bituminous paving mixtures of the dense-graded type, it is desirable to have the voids in compressed specimens above a minimum percentage and below a maximum percentage. The minimum percentage of voids should be such as to allow for increased densification by traffic loads and for expansion of the aggregate caused by temperature increase to avoid exuding bitumen on the surface and consequent reduction in skid resistance and stability. The maximum percentage of voids should be set at such a point as to insure sufficient density together with other desirable properties, such as stability and tensile strength."

It should be noted that the above statement recognizes two important facets of the present study: (a) density may increase and therefore there is a decrease in the volume occupied by a given quantity of material and (b) a decrease in stability (after reaching a certain

point) accompanies the increased densification.

Though the decrease in voids due to traffic compaction after a bituminous pavement is in place is universally recognized there appears to be little reference in the literature of the possibility of void changes during testing. Endersby (54) and Nijboer (49) recognize that volume changes take place but their discussions fail to present any significant data on the magnitude of these changes. Endersby (26) notes that for the mixes tested there was a decrease in density upon initial deformation and an increase in density thereafter. Lee and Markwick (31) observed: "Compacted bituminous materials increase slightly in volume when deformed." These authorities compare this action to that of granular materials known as 'dilatancy' and indicate that dilatancy is roughly proportional to deformation and is greater for mixes with low percentages of bitumen. Endersby (54) suggests the possibility of a critical void ratio for bituminous mixtures: "That is to say, a given mix, when compacted below a certain void content, will expand continuously during deformation. If the voids are above this point, the material will compact during deformation."

The effect of voids on stability is also the subject of much discussion. Hveem and Vallerga (45) state: "...there is evidence to show that when carried too far, reducing air voids in an asphalt pavement may result in other undesirable developments - loss in stability for example." In an earlier paper, Stanton and Hveem (25) present the following: "It is difficult to find evidence to prove that low void volume is the sole or even an essential measure of quality." However, Endersby (54) found: "...the change in stability resulting from

decreasing voids in a given mix, varies inversely somewhat as a straight line with voids." Neppe (1) discusses the effects of low void contents as follows: "It should however be realized that in a compacted mineral aggregate there is a continuity between individual voids by the interlocking of contiguous particles. If, however, the voids in the compacted mineral aggregate are overfilled, the surfaces of adjacent aggregate particles will be kept from making intimate contact and the stability of the mixture must inevitably be greatly reduced by the lubricating action of the binder."

Nijboer (49) draws two important conclusions regarding the stability of mixtures with very low percentages of voids. He indicates that for mixes with less than two percent voids a part of the externally applied pressure is carried by liquid phase of the mixture and that a very rapid decrease in resistance occurs as void contents are reduced to two-three percent. The first of these conclusions introduces the concept of pore pressures and effective stresses to the stress system of a bituminous mixture and implies that, in a fashion similar to soil, there would be a decrease in neutral stress and an increase in effective stress if time were provided for drainage. The reason for the limitation of this concept to mixtures of less than two percent voids is not apparent. The second conclusion is a direct outgrowth of the first for as effective stresses are decreased so would the shearing resistance of the mixture be reduced.

All of the preceding comments on the effects of voids in a mixture deal with their affect on stability which is the primary concern of this investigation. It should not be overlooked, however, that voids

influence other characteristics of mixes; in particular, the durability of the mixture. McLeod (55) draws attention to this aspect of voids considerations as follows: "The achievement of pavement durability requires careful consideration of the void properties of compressible bituminous mixtures." With high void contents, the mixture is subject to easy penetration of air and water and the deterioration of the mixture is hastened.

Influence of Specific Gravity Values

The evaluation of the percentage voids in a mixture is complicated by the weight-volume relationships that exist between aggregate and bitumen. It is common practice to design mixtures on the basis of weight and to compute the voids present in a compacted mixture by the difference in the volumes occupied by the solid and liquid ingredients and the volume of the compacted mixture. The adequacy of solutions established by this scheme of computations is directly related to the specific gravities of the ingredients that are used in the calculations. Ricketts, et al (56) summarizes the status of the various aggregate specific gravities:

> "The three conventional specific gravities have been used more or less interchangeably for bituminous aggregates, some agencies using bulk, some apparent, and some saturated surface-dry specific gravity; voids criteria of course must be consonant with the specific gravity used. When using aggregate of low absorption (less than 0.5%), the induced error is not too great; but when the more permeable aggregates are used the conventional specific gravities can provide unsatisfactory results."

The unsatisfactory results that are possible depend upon the absorption of bitumen by the aggregate. If the aggregate absorbs no bitumen, the

bulk specific gravity would yield correct values for the volume of aggregate; if the bitumen absorption is the same as water absorption the apparent specific gravity would yield correct results. However, in general, the bitumen absorbed by an aggregate is less than the amount of water the same aggregate would absorb but is some tangible value greater than zero. Therefore, voids computations using either specific gravity are incorrect. An aggregate specific gravity that is computed using a volume that provides an allowance for the bitumen absorbed would be the most correct value to use for void computations. The Bulk-impregnated Specific Gravity satisfies the need for a specific gravity measured as indicated (57).

A second approach to the problem of evaluating the effect of absorbed bitumen in specific gravity was developed by Rice (58)(59). In this procedure, a Maximum Theoretical Density of an uncompacted mixture of aggregate and bitumen is measured. This value may be used to compute an effective specific gravity of the aggregate which includes the effect of the absorbed bitumen. Because of the relative simplicity of the method many investigators prefer to work with this procedure.

Still other methods of accounting for the absorption of bitumen by aggregate are available (60)(61). Recent results of Hode-Keyser (62) indicate that the Rice method values are the most consistent and the most correct for a wide variety of aggregates of all of the proposed methods of evaluating the effect of absorbed bitumen on the specific gravity of aggregates.

All of the procedures indicated above provide means for computing a value for the percent absorption of bitumen by aggregate. McLeod (55)

recommends that the values for voids mineral aggregate and void content be computed on the basis of the ASTM Bulk Specific Gravity (63) and an allowance for the bitumen absorbed by the aggregate. This procedure defines the volume occupied by the aggregate components and reduces the volume occupied by the bitumen by the amount of bitumen absorbed by the aggregate. The net result is that the percent voids computed represent the space in a mixture that is free of solids or liquids as accurately as possible within the present knowledge of the subject. In addition, by this method of analysis, the asphalt content would be based on the amount actually available to fill the voids between mineral aggregate particles.

On the basis of the literature reviewed, it would appear that the objectives of this research constitute a necessary next step in the development of the technical knowledge of the performance of bituminous mixtures under triaxial compression loading.

MATERIALS AND PROCEDURES

This section of the report describes all details relating to the tests performed including a description of the materials used, characterization test results, the results of a study on sample fabrication procedures, the equipment developed for the investigation, and the procedures followed in the triaxial compression testing of all samples.

Materials

The bituminous concrete mixture utilized for this study was chosen to be typical of such mixtures in use at the present time by various agencies. It was not intended that the mixture satisfy the specifications of any one organization. However, the mixture does satisfy, in general, the requirements of the State Highway Department of Indiana for Hot Asphaltic Concrete Surface, Type A (64).

The composition of the aggregate blend used is shown in Table 1, Table 2, and Figure 1. In Figure 1 the gradation of two other mixtures are shown for comparison purposes.

The coarse aggregate (percent retained on No. 4 sieve) utilized in this study was a crushed and washed limestone from central Indiana (laboratory No. 67-2S). The limestone source is located in the St. Genevive formation and is of Mississippian age. The limestone is characterized by a low porosity and a fine-grained texture. The values of specific gravity established by ASTM Method C-127 (66) are as follows:

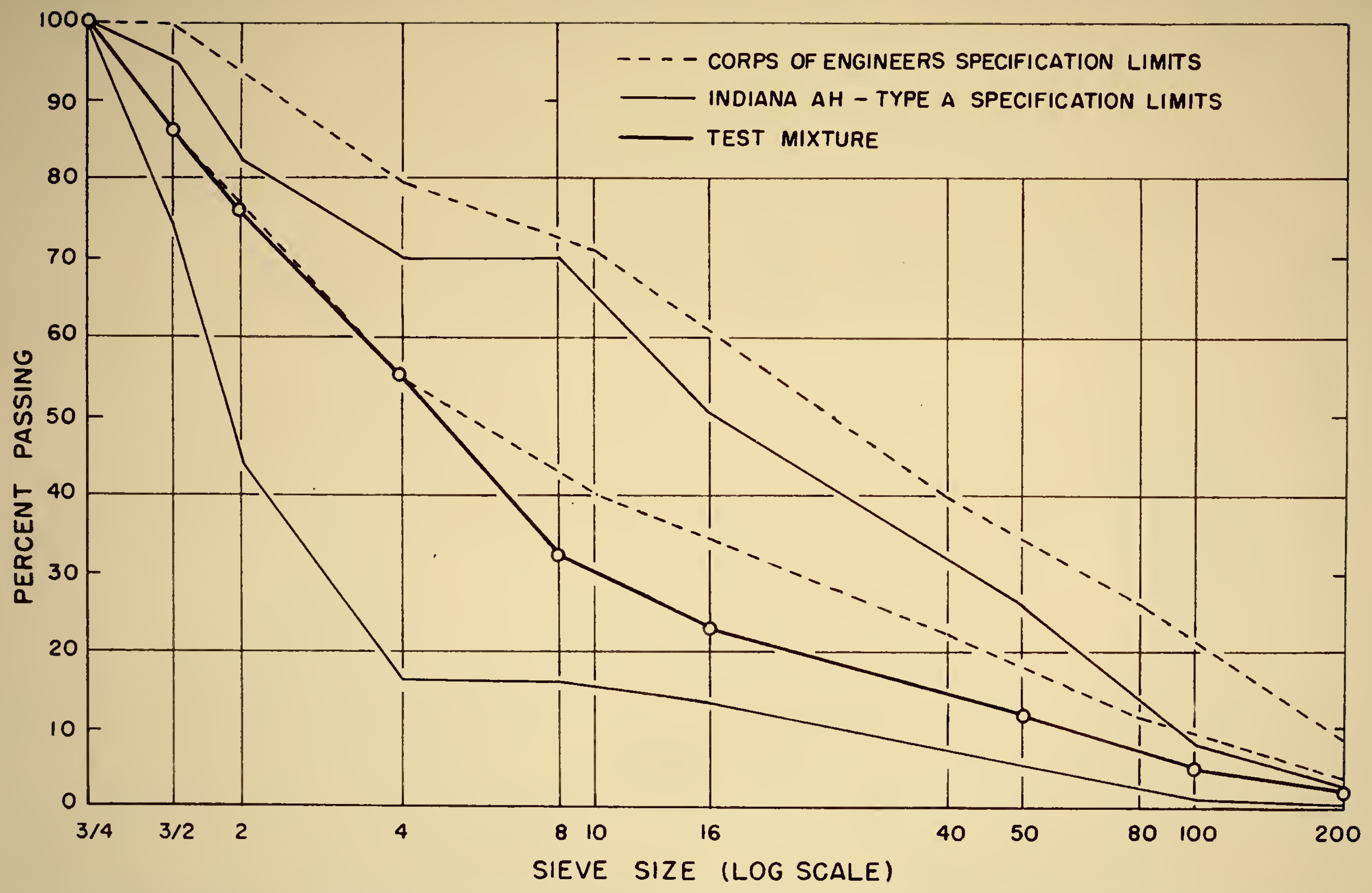


FIGURE I. AGGREGATE GRADATION FOR TEST MIXTURE

TABLE 1

Aggregate Gradation Data

Sieve No.	Percent Passing		
	Indiana Type A (64)	Corps of Engineers 3/4 Maximum Surface Course (39)	As Used
3/4	100	100	100
1/2	75-95	86-100	86.0
3/8	45-83		76.0
4	16-70	55-80	55.0
8	16-70		32.0
10		40-66	
16	13-50		22.5
40		22-40	
50	5-25		12.0
80		12-26	
100	1-8		5.0
200	0-3	4-9	2.0

TABLE 2

Aggregate Gradation of Test Mixture

Passing Sieve No.	Retained Sieve No.	As Used, %
3/4	1/2	14.0
1/2	3/8	10.0
3/8	4	21.0
4	8	23.0
8	16	9.5
16	50	10.5
50	100	7.0
100	200	3.0
200	-	2.0

Bulk Specific Gravity	2.660
Apparent Specific Gravity	2.700
% water absorption	0.63%

The fine aggregate (percent passing the No. 4 sieve and retained on the No. 200 sieve) was obtained from two sources. The portion passing the No. 100 sieve was a very uniform dune sand obtained near Monon, Indiana. The remainder of the fine aggregate was obtained from a local river terrace deposit (laboratory sample No. 79-1). The specific gravity values obtained by using ASTM method C-128 (63) were:

	No. 79-1	Dune Sand
Bulk Specific Gravity	2.574	2.595
Apparent Specific Gravity	2.715	2.656
% water absorption	2.01	0.88

Portland cement was used for the filler (percent passing the No. 200 sieve). The material used was a Type I cement with a specific gravity of 3.15 and satisfied all requirements of ASTM designation C-150 (65). Portland cement was chosen for the filler to insure a supply of uniform material meeting the desired gradation requirements and for its ease in handling in the laboratory.

As the question of voids in the compacted mixture was critical to the study, it was desirable to obtain the percentage of bitumen absorbed by the aggregate blend. This problem was approached by two different methods: (a) Bulk-impregnated Specific Gravity (56) of the aggregates and (b) computed from the Maximum Theoretical Density (59) of the aggregate blend. Using the procedures outlined in the references indicated for the former process, the following values were obtained:

	Bulk-impregnated Sp. Gr.	% bitumen absorbed
Coarse Aggregate	2.671	0.15
Fine Aggregate	2.642	1.01

The filler material was assumed to absorb no bitumen. Using the above data, an average bulk-impregnated specific gravity of the blend of 2.664 was computed and a value of bitumen absorbed for the aggregate blend was found to be 0.61%.

With the Maximum Theoretical Density (Rice) procedure, an average effective specific gravity of the aggregate blend was computed to be 2.672 and the percent bitumen absorbed was found to be 0.72%.

It was considered that the values established by the Maximum Theoretical Density procedure were more apropos to this study as they reflect the measured characteristics of the blend whereas the Bulk-impregnated Specific Gravity procedure established these values by computation from determinations on the aggregate fractions. With the excellent correlation of the results of the two procedures, the choice of test values for use in design is relatively unimportant.

An 85-100 penetration grade asphalt cement was provided by the Texas Company from the Port Neches refinery for use in this study. Test results on the asphalt are presented in Table 3.

The aggregate blend chosen and the range of asphalt contents considered for the testing program were evaluated for general suitability as an asphalt paving mixture by means of the Marshall (40) and Stabilometer test procedures (41). The results of these characterization tests are shown in Table 4.

TABLE 3

Results of Tests on Asphalt Cement

Penetration, 100 g., 5 sec., 77°F	93
Penetration, 100 g., 5 sec., 32°F	21
Softening Point, Ring and Ball, °F	116
Ductility, 77°F, cm.	200+
Flash Point, Cleveland Open Cup, °F	590
Solubility in CCl_4, %	99.86
Spot Test	Negative
Specific Gravity, 77/77 °F	1.033

TABLE 4

Results of Characterization Tests

Marshall Method Results

Asphalt Content % by weight of mix	Stability lbs.	Flow 1/100 in.	Unit Weight lb/cu.ft.	Voids %	Voids Filled %
3.75	1105	11.5	145.7	6.2	58.0
4.25	1165	13.3	147.3	4.6	69.0
4.85	1125	13.8	149.1	2.6	82.0
5.19	1232	14.3	149.5	1.8	87.5
5.74	1050	15.9	149.1	1.1	92.5

Stabilometer Method Results

CKE, fine aggregate	2.2
Oil Equivalent, coarse aggregate	3.3
Surface Area, sq.ft/lb.	18.5
Optimum Asphalt Content, % by weight of aggregate	5.2

Asphalt Content %					
by weight of mix	by weight of aggregate	Stabilometer Value	Unit Weight lb/cu.ft.	Voids %	Voids Filled %
4.06	4.23	45	150.4	4.7	67.5
4.27	4.46	43	151.4	3.9	72.7
4.77	5.01	37	152.7	2.4	83.0
5.32	5.63	31	152.3	1.7	88.5

Batching and Mixing

The aggregates used were batched by component fractions according to the blend formula. Batching was accomplished with cold dried aggregates using a torsion balance sensitive to 0.1 g. The blended aggregates and asphalt for a batch were heated separately to 300°F ± 10° in a Peerless gas oven. At the same time the mix ingredients were being heated, the mixing bowl, paddle and miscellaneous spoons and spatulas were heated. The combining of the aggregate and asphalt for a batch was accomplished using a Toledo scale (10 Kg capacity, 1 g direct reading). A tare weight of the mixing bowl was recorded, the hot aggregate was added to the bowl, and the combined weight noted. The difference in this weight and the tare weight of the bowl established the actual weight of hot aggregate used in the batch and was the weight used in all asphalt content computations. The heated asphalt cement was added with the mixing bowl and aggregate on the Toledo scale. The direct reading fan of the scale permitted control of the amount of asphalt added to the nearest one gram. A check on the weight of asphalt cement used was made by noting the difference in weight of the pan of asphalt before and after adding the material to the aggregate. A typical mix record is shown in Appendix B.

With the desired quantities of hot materials in the bowl, mixing was accomplished using a Hobart electric mixer (Model N-50) modified with a special mixing paddle and scraper. The mixing was continued for two minutes.

After mixing, the batch was placed in a shallow, flat-bottomed metal pan which had been heated to 140°F. Care was exercised to obtain, as

nearly as possible, all material from the mixing bowl and mixer paddles. As two batches were necessary to provide the material necessary for the fabrication of one specimen, the above mixing procedure was repeated while the first batch was stored in a 140°F oven (Electric Hotpack Model 14-12) to prevent excessive cooling. The two batches were combined and hand-mixed in an effort to obtain a single mass of uniform material.

The mixed material was cured in the shallow pan for a period of 18-20 hours in a forced-draft oven at 140°F prior to compaction.

Compaction

Three methods of compaction were available for the formation of specimens of the size desired for this study. Dynamic compaction would have required an intensive study of the relationship between height of fall, weight of tamper, number of blows and number of layers in order to arrive at a satisfactory procedure to fabricate an eight inch high specimen. The problem of developing a reasonably uniform density throughout the height of the specimen was a major objection to static compaction. The third method, kneading compaction, seemed to offer the best possibilities for providing a desirable specimen. In addition, this method of compaction was consonant with the present trend of thought regarding the formation of a specimen most nearly like the pavement prototype.

The principles of kneading compaction have been discussed frequently as has been the operation of the kneading compactor (66)(44)(38). In general, the compaction process is accomplished as follows: Material is fed into the mold which is mounted on a rotating table. The compaction foot, a pie-shaped segment with an area approximately one-fourth of that of the mold, is activated by a combination mechanical and

hydraulic-pneumatic control system. The foot moves downward against the sample until a pre-set load is built up. The load is maintained for a short interval of time and then released. A load cycle requires two seconds to complete. Means are available for adjusting the foot pressure over a wide range of pressures. Material may be placed in the mold in layers or by a continuous flow process during compaction.

The kneading compactor model available for use is shown in Figure 2. It was constructed by the August Manufacturing Company of Oakland, California. A special mold, Figure 3, suitable for the formation of specimens eight inches high by four inches in diameter was constructed in the shops of the School of Civil Engineering.

The general scheme of compaction considered for this study consisted of placing a portion of the sample mixture into the mold, rodding with a three-quarter inch diameter bullet-nose rod, and applying a given number of tamps after which the remainder of the mix was fed into the mold while compaction was continuing. A final number of compacting tamps on the surface of the sample completed the sample fabrication. The problem was to establish the most desirable combination of tamps, foot pressures, etc. for the fabrication of specimens from the mixture to be used for the study.

The criteria for a suitable specimen was considered to be uniform density and asphalt content throughout the height of the sample. Uniformity of these two measures implies a uniform distribution of voids throughout the specimen. Void uniformity was considered desirable in light of the purposes of the study. To evaluate the uniformity of asphalt content and density, each specimen made for purposes of developing

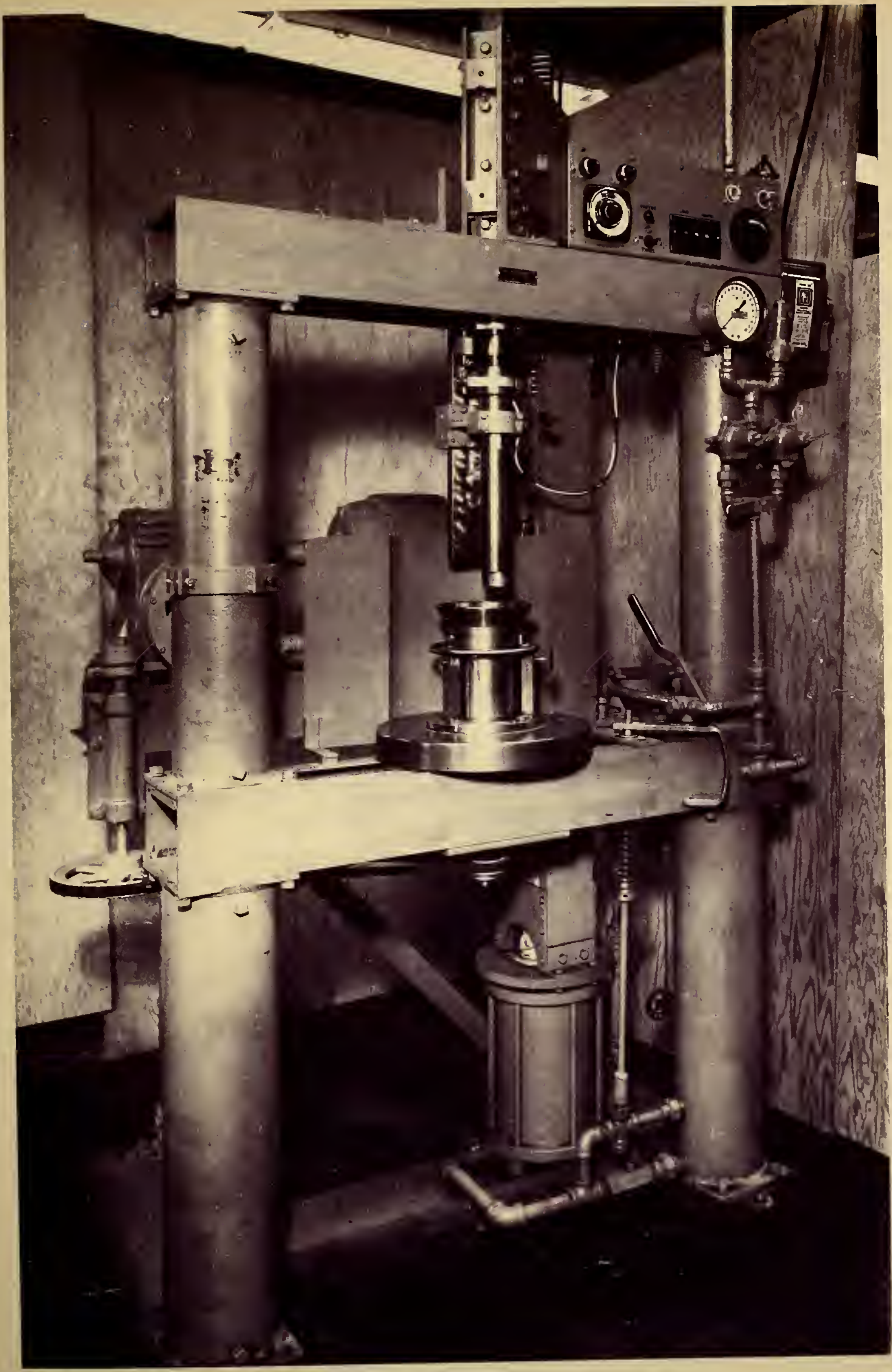


FIGURE 2. KNEADING COMPACTOR

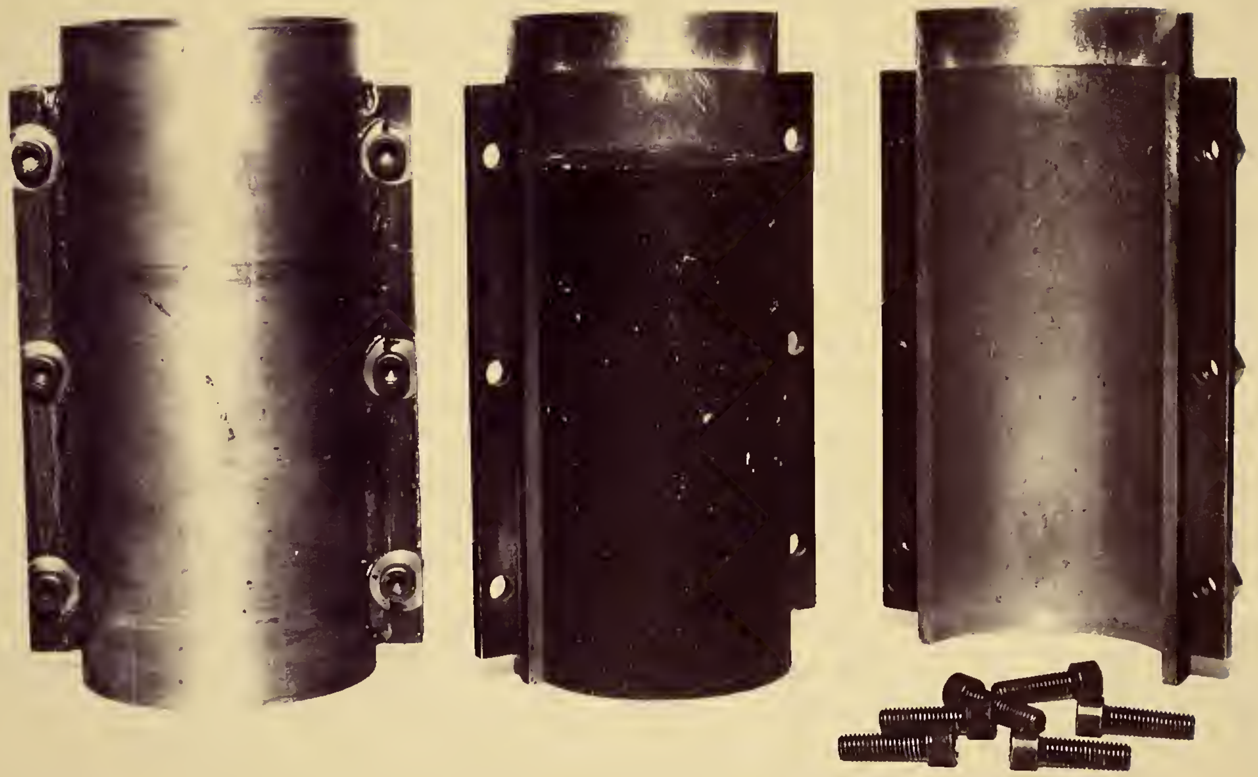


FIGURE 3. MOLD FOR TRIAXIAL TEST SPECIMENS

a compaction procedure was cut into three sections perpendicular to the axis of the sample. Each section was measured, weighed, and the volume computed by displacement in water using the principle of Archimedes and by computation using the specimen dimensions. The sections were then treated for the extraction of asphalt using ASTM Method D-1097 (67).

Absolute uniformity of density or asphalt content was considered to be unattainable from a practical standpoint. It was decided, therefore, to strive for a procedure which yielded density values of the sections within one percent of the average of the three sections and an asphalt content of the sections within three-tenths of one percent of the average asphalt content of the three sections as established from extraction test results.

Numerous trial compaction procedures were investigated. The procedure finally settled upon was as follows:

1. The mixture was removed from the 140°F curing oven after 18-20 hours of curing and was remixed by hand.
2. The mixture was split into two approximately equal parts and each part brought to a temperature of 230°F ± 10°.
3. One-quarter of the mixture was introduced into the preheated mold which was placed on the rotating table of the kneading compactor.
4. To reduce the tendency for surface voids to develop in the lower portion of the sample, the first one-quarter of the mixture was rodded with a 3/4 inch bullet-nose rod. Twenty tamps were applied to the center of the mix and twenty around the periphery of the mix.

5. The material in the mold was subjected to 15 tamps of the tamping foot under the desired and pre-set foot pressure.
6. The remainder of the sample was introduced into the mold while the compactor was in operation. One-third of this portion (one-quarter of the total sample) was added gradually over each of three two-minute periods. All material was stored in the heating oven at a temperature of 230°F ± 10° until ready for compaction. This procedure permitted the application of 180 tamps on three-fourths of the original sample while this quantity was being added to the mold.
7. Two minutes of tamping (60 tamps) was applied to the surface of the entire sample.
8. The mold was removed from the kneading compactor and the sample subjected to a double plunger static load of 12600 pounds as a leveling load. This load was applied at a rate of 0.025 inches of deformation per minute and was released immediately upon reaching the load specified. The loading was applied by the use of a Riehle Testing Machine, 50,000 pound capacity, and having a variable speed drive.
9. The mold was allowed to cool in air for approximately 30 minutes, at which time the mold was disassembled and the specimen removed.
10. Each specimen was marked for identification. The bulk density of each was determined from the dimensions of the specimen for purposes of establishing the uniformity of the specimens.

11. All specimens were stored in a constant temperature atmosphere of 77°F until time for testing.

Tables presented in Appendix C show typical density and extraction asphalt content results for specimens made by the foregoing procedure at asphalt contents by weight of mixture of 2.8, 4.0, and 5.0 percent and at 150, 250, and 400 psi compaction foot pressures. It may be noted in all cases that the uniformity of density was well within the limit sought. In the majority of cases, the density of the sections varied from the average by less than one-half of one percent. The uniformity of asphalt content that was desired was attained in all specimens.

Controlled Temperature Room

In order to eliminate the variable of temperature and its effects upon the tests performed in this study, a controlled-temperature atmosphere was considered necessary. A commercially available unit marketed by The Electric Hotpack Company, Inc., Philadelphia, Pennsylvania, under the name Controlled Environmental Room, Model 8316, was found to satisfy all requirements of the project and was obtained for use.

Figure 4 is a photograph of the exterior of the controlled-temperature room while Figure 5 shows the interior of the room with all testing equipment in place. Figure 6 shows a close-up view of the control panel and recording unit on the exterior of the room.

The room obtained was 6 x 8 x 7 feet and was assembled in the laboratory from four prefabricated sections. All portions of the room in contact with the surrounding atmosphere contained three inches of precompressed glass wool as insulation. Temperature control was provided

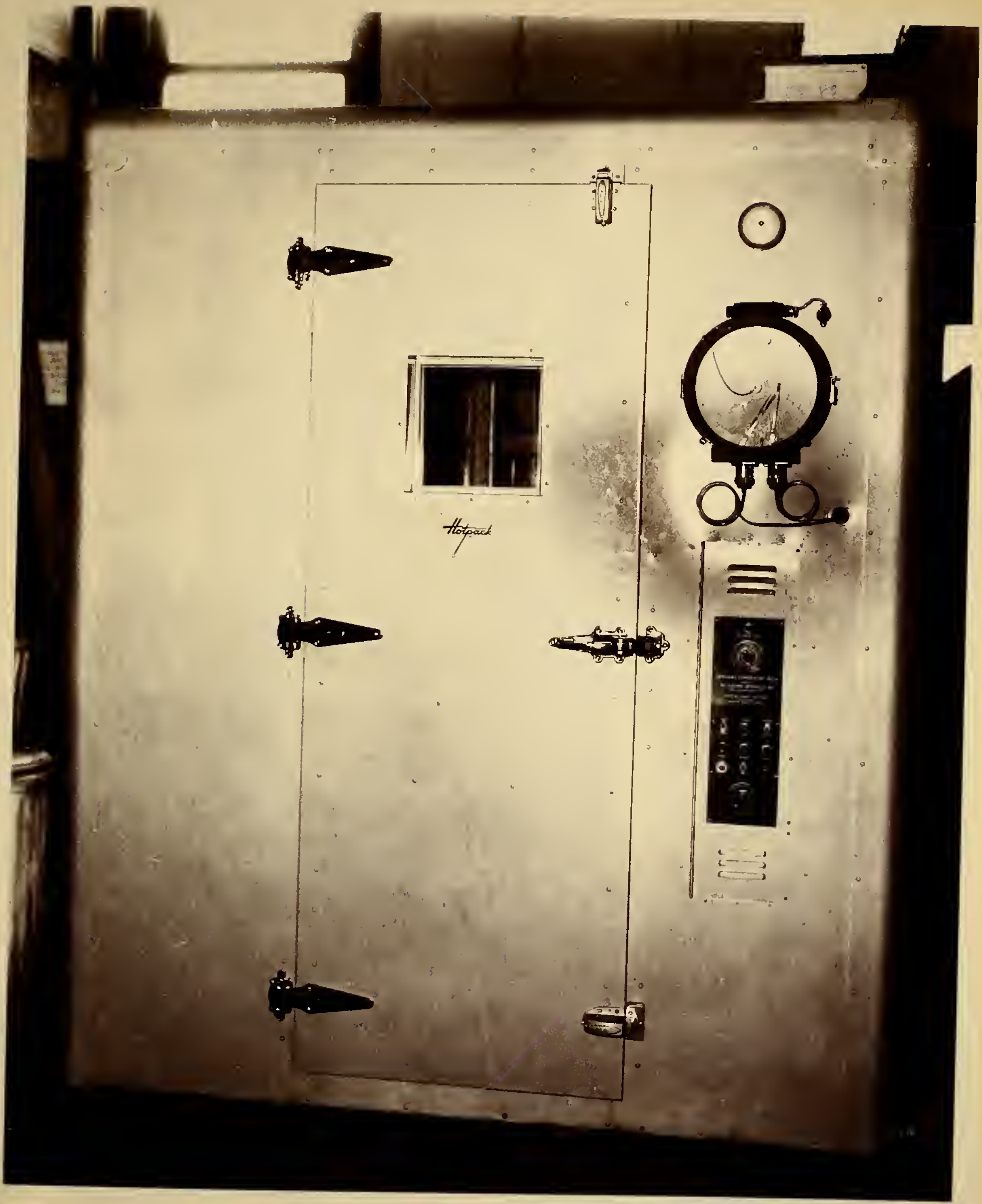


FIGURE 4. CONTROLLED TEMPERATURE ROOM—EXTERIOR VIEW

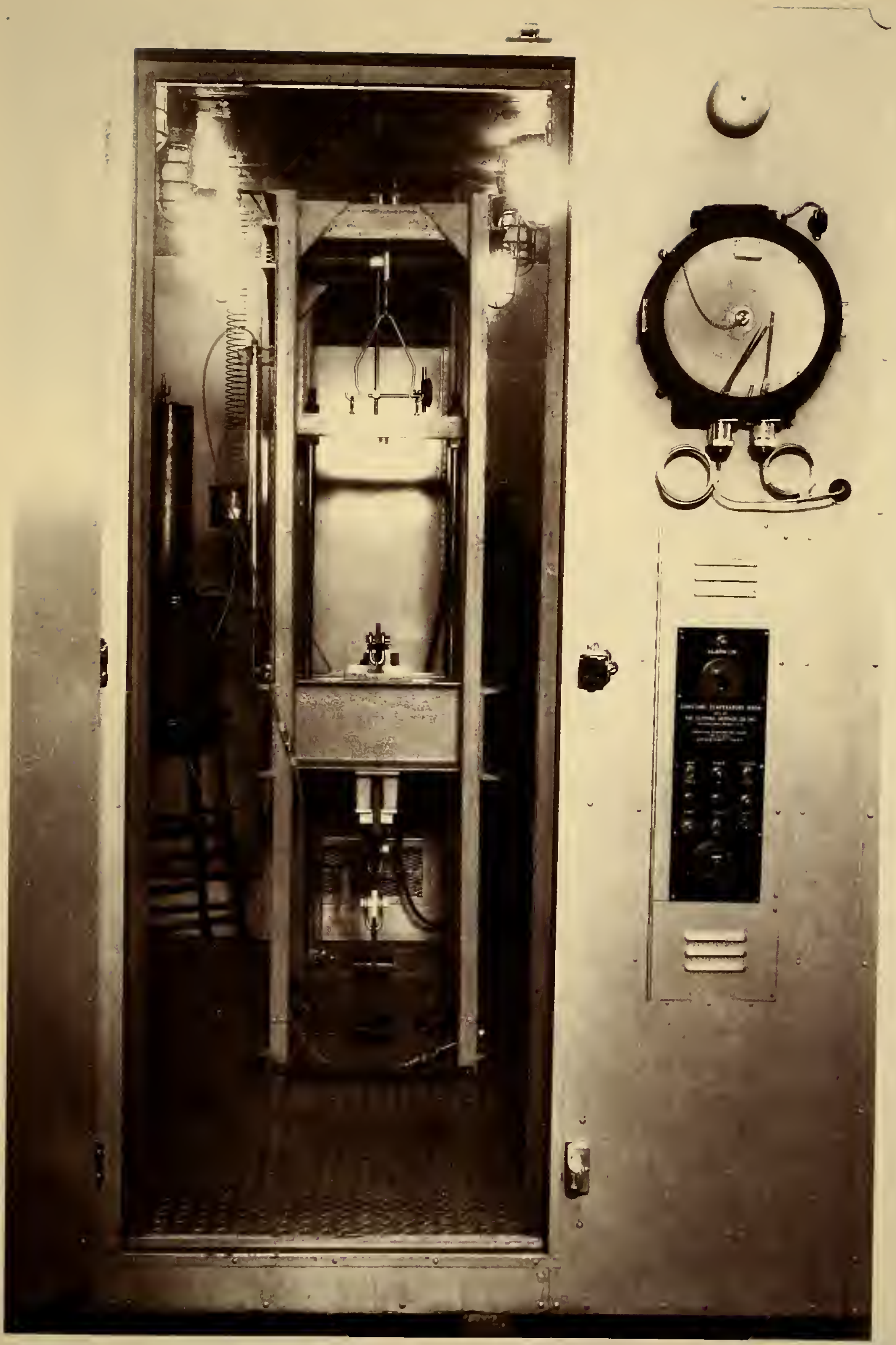


FIGURE 5. CONTROLLED TEMPERATURE ROOM—INTERIOR VIEW

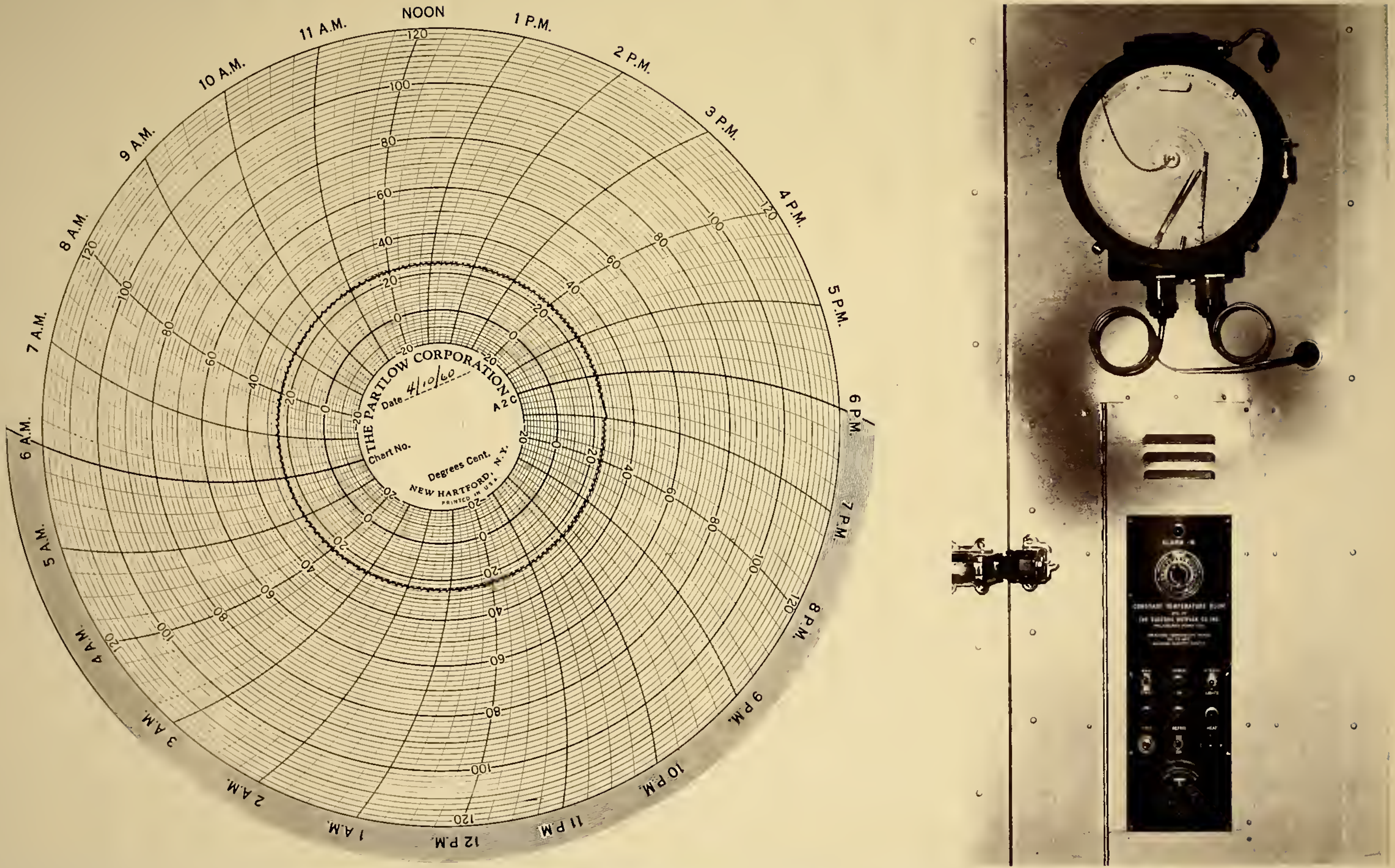


FIGURE 6. CONTROL PANEL, RECORDING UNIT AND TYPICAL TEMPERATURE RECORD FOR CONTROLLED TEMPERATURE ROOM

through Partlow thermostatic controls operating heating coils and a refrigeration unit. Humidity control was an integral part of the room but was not utilized for this study. The room was rated electrically as operating on 230 volt, single-phase AC current and requiring approximately 4800 watts of power.

The recording unit provided a continuous record of the room's temperature through a 24 hour period. Tests conducted during the early period of operation of the room indicated that the recorded trace was an accurate measure of the dry bulb temperature of the room at the sensing element. A series of accurate thermometers were placed at points of interest throughout the room and observed carefully for long periods to note the variation of temperature within the room. Average air temperatures at the triaxial cell and at the volume measuring device were observed to be 0.3°C above the temperature at the sensing element. Temperature readings taken every minute on all thermometers indicated a cycling of temperature from approximately 1°C above the average to approximately 1°C below the average with the cycling occurring approximately once every ten minutes. A typical set of observations are presented in Appendix D.

Observations on the water temperature within the elements of the volume measuring device showed a maximum variation of ± 0.1°C from the temperature at the sensing element. In summary, all temperature observations indicated that the average temperature at any point within the room was within the desired maximum temperature range of ± 0.5°C. A typical temperature record, as made by the Partlow recorder, is shown in Figure 6.

Volume Measuring Device

The basic idea which resulted in this study was a belief that bituminous-mixture specimens change volume upon the application of various test loadings. It was necessary, therefore, to develop a method of measuring the specimen volume or change in volume at various stages during the performance of a triaxial test.

Several schemes were considered to provide a procedure for the measurement of the volumes of the specimens under test conditions. The most promising method was one adopted from a procedure used by the Soil Mechanics Laboratory of the University of London and reported by Bishop and Henkel (23). Figure 7 is a photograph of the device used in this study while Figure 8 is a diagrammatic sketch of the volume measuring device that also shows its relationship to the triaxial cell and the pressure source. The key following these figures identifies the principal parts of the apparatus (Table 5).

The basic principle upon which the volume measuring device was operated is simple. Any change in the volume of the sample was reflected by a change in the volume of the liquid in the triaxial cell. The magnitude of this volume change was measured by a change in the level of the mercury column in the measuring tube. A differential head of mercury between the measuring tube and the mercury reservoir tube was prevented by mounting the latter tube on a calibrated spring that changed length as the amount of mercury in the reservoir tube changed.

The volume measuring device was operated as follows: The triaxial cell was filled with the confining liquid, water in this case, with care taken to remove all air bubbles from the cell. At this point, the water

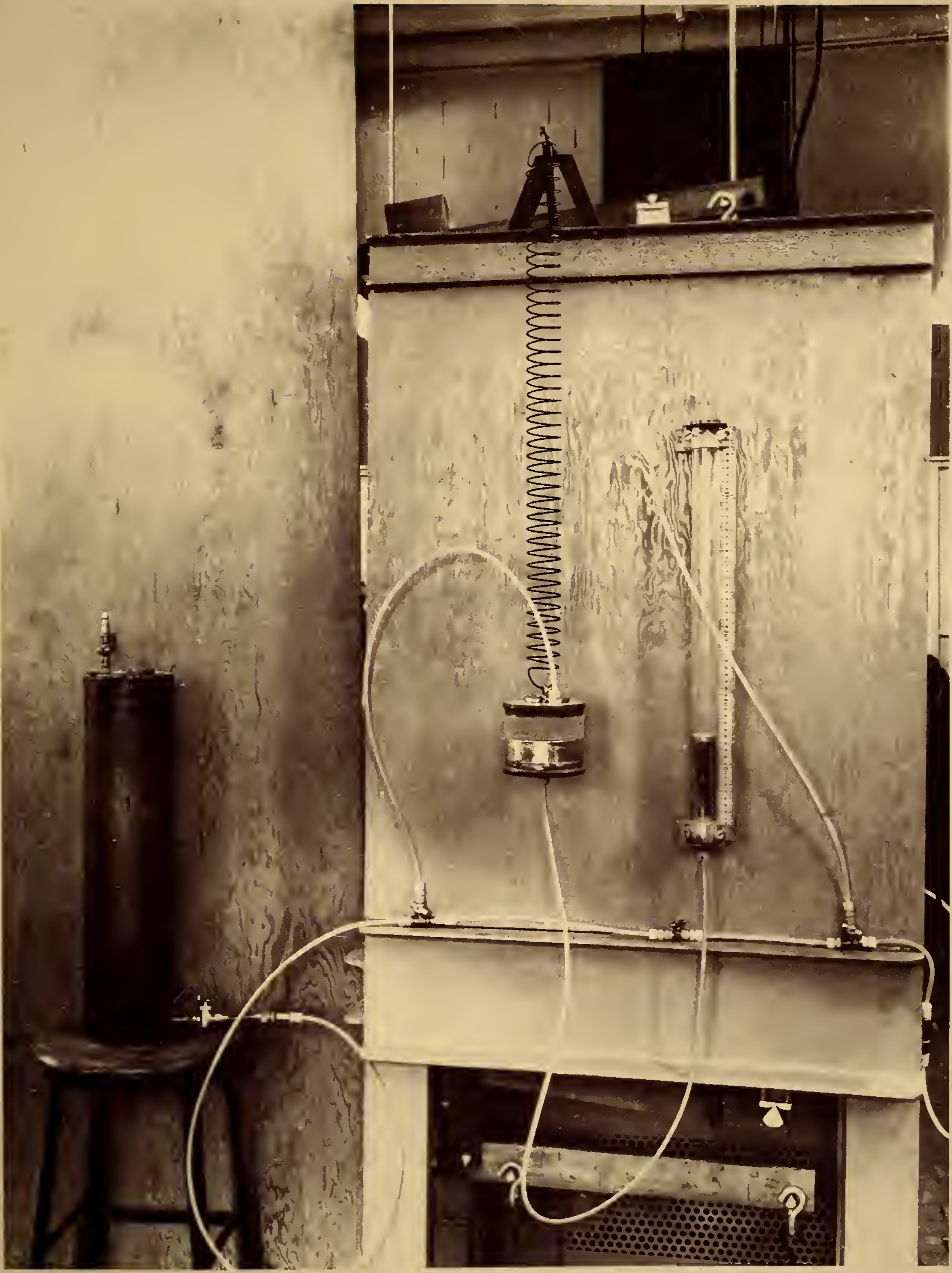


FIGURE 7. VOLUME MEASURING DEVICE

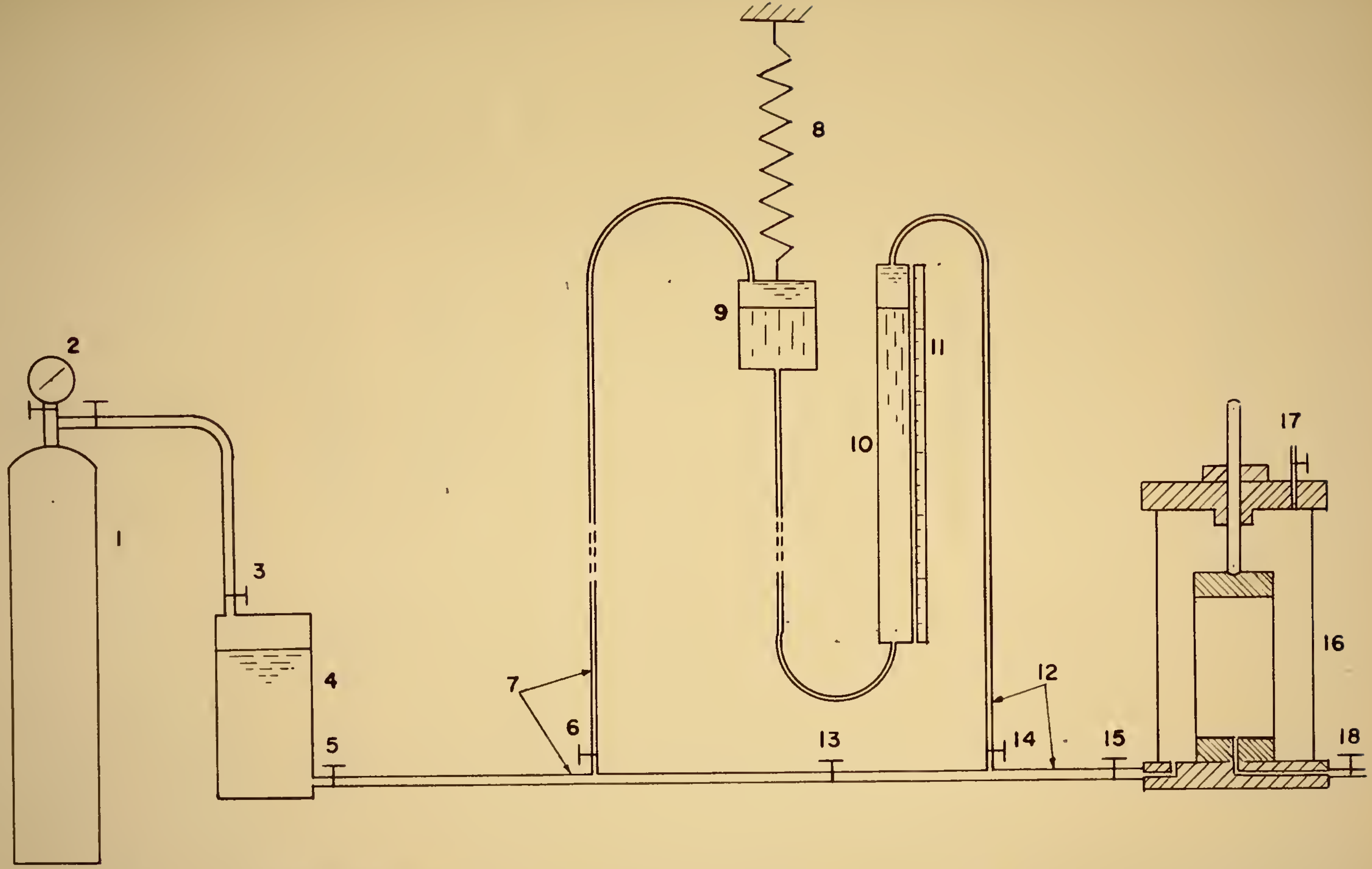


FIGURE 8. DIAGRAMMATIC SKETCH OF VOLUME MEASURING DEVICE

TABLE 5

Key to Details of Figure 8

1. Nitrogen tank
2. Pressure regulator
3. Quick release pressure coupling
4. Water reservoir tank
5. Water reservoir tank outlet valve
6. Mercury reservoir control valve
7. Polyethylene plastic tubing (1/4 inch O.D.)
8. Calibrated spring
9. Plastic mercury reservoir tube
10. Plastic measuring tube
11. Metric scale
12. Polyethylene plastic tubing (1/4 inch O.D.)
13. Bypass valve
14. Measuring tube control valve
15. Triaxial cell water inlet valve
16. Triaxial cell
17. Triaxial cell air escape valve
18. Sample drainage valve

in the cell was continuous to the water in the reservoir while the flexible tubing and the measuring tubes were completely filled with water and mercury. Valves were adjusted so that a pressure applied to the water reservoir was transmitted through the flexible tubing to the liquid in the cell. Upon the application of pressure to the cell liquid (the confining pressure of the test), a change in the level of the mercury in the measuring tube took place. This change reflected the increase in the volume of the cell liquid that resulted from the lateral pressure forcing the impervious membrane into the surface voids of the specimen. The change in volume of the specimen with time under any load application to the specimen caused a change in the level of the mercury in the measuring tube. For example, if the specimen compacted or decreased in volume under load, water entered the cell from the flexible tubing and the mercury level of the measuring tube moved upwards an amount equal to an equivalent volume. Simultaneously the mercury level in the reservoir tube dropped. The decrease in weight of the reservoir tube caused the calibrated spring to shorten until a uniform level of mercury existed between the two mercury tubes.

The metric scale located adjacent to the measurement tube permitted a reading of the change in level of the mercury to the nearest one millimeter which corresponded to a volume change of 0.497 cubic centimeters. A volume change of this magnitude is equivalent to a change in volume of the specimen of approximately 0.03 percent. It should be noted that any observed change in mercury level must be corrected for any variation in temperature, for the change in volume of the cell under the applied confining pressure, and for the volume displaced by the movement of the piston.

Temperature changes were minimized by the use of a controlled temperature atmosphere in which to perform all tests. A maximum temperature variation of one-half degree Centigrade was established for the test location. With these temperature conditions the maximum possible variation of the volume of the cell liquid and the volume of the cell has been computed to be ± 0.255 cubic centimeters or a variation in mercury level of approximately one-half millimeter.

The change in the volume of the cell and the volume measuring device under the applied confining pressure was established experimentally by calibration tests. A steel dummy specimen the same size as a bituminous specimen was used to simulate it and the cell was assembled as for a routine test. The changes in volume of the apparatus under the various confining pressures to be used were observed and recorded as calibration factors for the assembly. The change in volume of the assembly under a confining pressure occurs almost instantaneously upon the application of the pressure. Little change in volume occurred with time under prolonged application of the confining pressure.

The entrance of the piston under load into the triaxial cell caused a continuing displacement of the cell liquid that resulted in a decrease in the level of the mercury in the measuring tube. Computations based on the diameter of the piston and the deformation of the specimen permitted the evaluation of a correction factor which was applied to apparent volume changes.

Observations of temperature variation within the controlled-temperature atmosphere, test measurements of the change in volume of the apparatus under the various confining pressures used and with time, and

the magnitude of the volume changes due to piston displacement are presented in Appendix D.

Triaxial Testing Equipment

Figure 5 and Figure 9 are general views of the triaxial testing equipment used in this study. The former figure shows a front view of the testing frame in position within the controlled-temperature room while the latter figure is a close-up of the loading crossbar of the frame with the triaxial cell in position. Figure 10 shows a section of the triaxial cell and should be used in conjunction with Table 6 which serves as a key to the major elements of the cell.

The loading frame was constructed in the shops of the School of Civil Engineering. In general, it is similar to a consolidation frame available through Soiltest, Inc., but with dimensions increased to permit testing of large triaxial specimens. Loads were applied as dead weight increments to the hangers at the underneath side of the frame. The rear hanger provided a 10:1 load ratio to the piston and was used exclusively for providing weight to balance the cell pressure exerted on the piston. The forward hanger permitted a 40:1 load ratio to the piston. This hanger was used for all load increments. Counterbalances within the lever system insured that no loads from the frame itself were applied to the specimen during the test operation. A mechanical jacking device was utilized to maintain the loading arm horizontal during testing regardless of the deformation of the specimen. Load increments of a nominal 50, 20, and 10 psi to a four inch diameter specimen were fabricated from plate steel for use in load

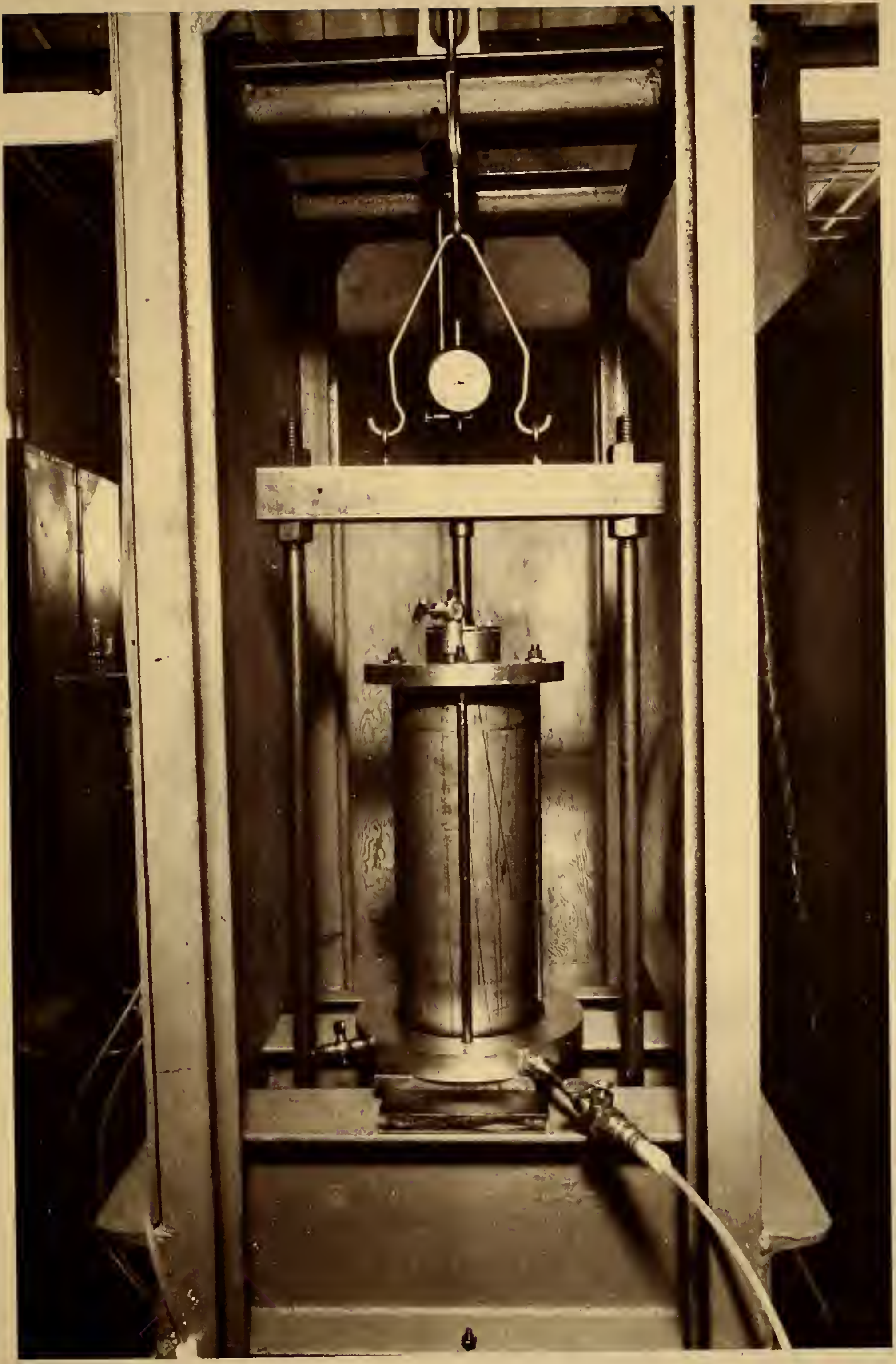


FIGURE 9. TRIAXIAL CELL IN LOADING FRAME

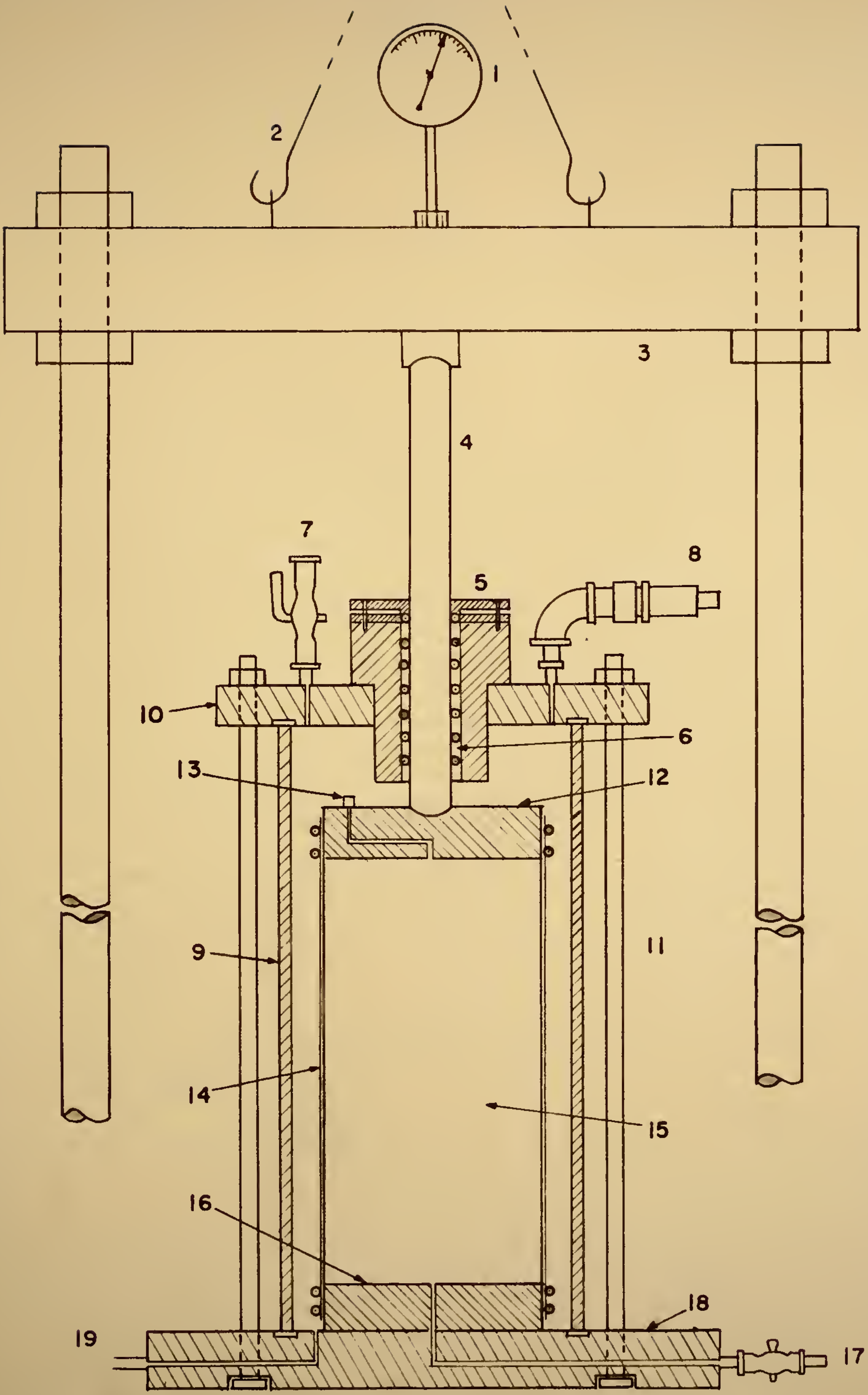


FIGURE 10. DETAILS OF TRIAXIAL CELL

TABLE 6

Key to Details of Figure 10

1. Dial gage, 0.01 mm.
2. Loading head support
3. Loading head
4. Piston
5. O-Ring seal assembly
6. Ball-bushing seal assembly
7. Air escape valve
8. Pressure relief valve
9. Cylindrical wall of triaxial cell
10. Upper head assembly
11. Tie rods
12. Upper loading cap
13. Upper drainage connection (sealed)
14. Rubber membrane with O-Ring seals
15. Specimen
16. Bottom loading cap
17. Bottom drainage connection with valve
18. Bottom head assembly
19. Water inlet connection

application. Axial deformation of the specimen was measured with a 0.01 millimeter dial gage suspended from a top crossbar of the frame and with the gage pointer in contact with the loading crossbar and positioned over the center of the piston.

The triaxial cell was modified from one available in the Bituminous Mixtures Laboratory from previous work done by Oppenlander (4). The top and base of the cell were of aluminium and the cylinder was brass. Compression loads were applied to a loading cap resting in position on the specimen by means of a hardened steel piston. The loading cap was recessed to receive the spherical end of the piston. The piston was centered through a vertical ball bushing and grease seal in the center of the cell. Preliminary tests showed the grease seal unable to hold the higher lateral pressures without appreciable leaking. Consequently, an O-Ring seal was installed above the grease seal. Adjustment of thumb screws permitted a cover plate to be forced against the O-Ring which expanded against the piston to provide a water-tight seal. The remainder of the cell details are apparent upon reference to Figure 10 and to the identification key shown in Table 6.

The use of the O-Ring seal was successful in eliminating leakage but it was apparent that this procedure materially increased frictional resistance to the piston. In order to evaluate piston friction, a proving ring was set up inside the cell which was assembled using a clear lucite cylinder. The use of known loads on the hanger permitted a computation of the load applied to the piston. Evaluation of the load reaching the proving ring was possible through the calibration charts of the proving ring. The difference between these loads was

assumed to be friction. A plot of load applied versus friction was developed and used to correct all applied loads in the computation of test results. This plot is presented in Appendix D.

Triaxial Test Procedure

All testing was performed using the 'open' or constant-lateral-pressure triaxial test procedure. The following sub-sections describe the details of the preparation of specimens for testing, the loading procedures used, the technique utilized in disassembling the apparatus upon completion of a test, and an evaluation of the test procedure.

Preparation of Specimens for Testing

The preparation of a specimen for testing was a time-consuming operation requiring careful attention to details. The specimen to be tested was removed from storage in the controlled-temperature room and carefully centered on the bottom loading cap of the triaxial cell. The top loading cap was placed on the sample. The circumferential surfaces of both caps were covered with a thin film of Dow-Corning Hi-Vacuum Silicone Grease. A thin rubber membrane, available from Soiltest, Inc., in a size adequate for four inch diameter by eight inch high specimens, was placed over the specimen utilizing a membrane stretcher for the operation. The membrane was smoothed carefully to the surface of the specimen and both loading caps. The membrane was sealed to the loading caps by the use of two O-Rings (2-7/8 inch diameter) at each end. A vacuum was applied to the specimen through the lower drainage connections. If the membrane held the vacuum it was assumed to be a leak-free set-up and the assembly of the cell continued. A photograph of a

partially completed test assembly is shown in Figure 11.

This procedure was used for all samples except those to be tested with a 60 psi confining pressure. Experience indicated that one membrane was not capable, in general, of withstanding this level of confining pressure without rupturing or leaking. As a result, double membranes were used for all 60 psi confining pressure tests. The second membrane was placed over the first one and secured by an additional two O-Rings at each end. The double membrane procedure performed well for the high-confining-pressure tests.

The cell assembly was completed by putting the cell cylinder, the connecting rods, and the top including the piston into place. The connecting rods were fastened by tightening the rod nuts uniformly and the piston was pressed firmly into place in the spherical seat of the upper loading cap. The assembled cell and specimen were placed in position within the triaxial frame.

Once the cell and specimen were placed in the loading frame the test progressed in a routine manner. The loading bar was brought into contact with the piston and adjusted to a balance in this position. The hanger loading arm was leveled and the deflection dial adjusted in place on the loading bar. The volume measuring device was attached to the water inlet and the cell was filled with water. During the filling operation, the valves to the measuring tube and mercury reservoir tube were closed. Gravity flow was not sufficient to fill the cell; therefore, a slight (5 to 8 psi) pressure was applied to the top of the reservoir tank. When water first flowed from the air escape valve, the water inlet valve and the water reservoir outlet valves were closed and

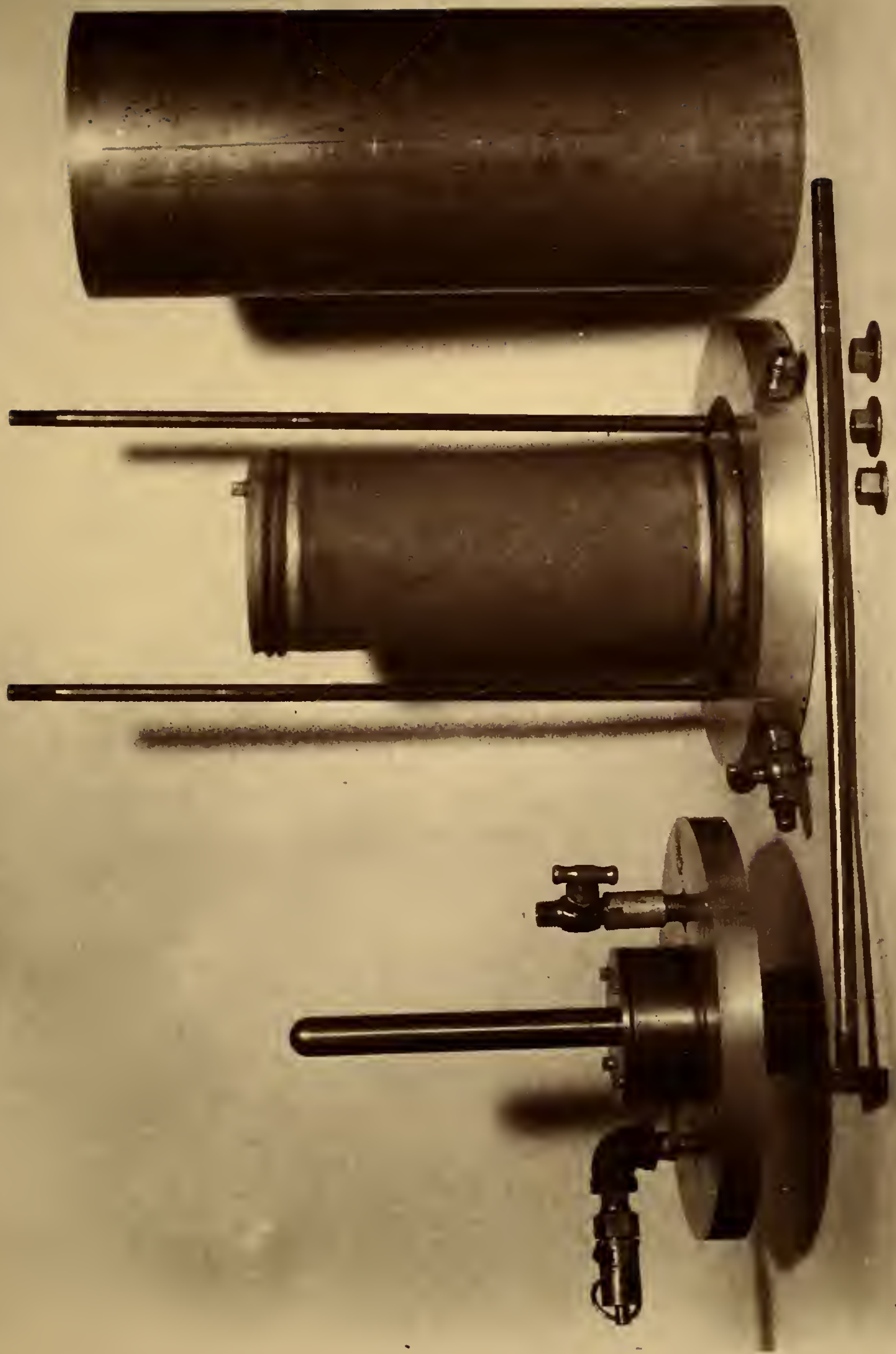


FIGURE II. ASSEMBLY OF TRIAXIAL CELL

the pressure connection to the water reservoir was disconnected. To check on the removal of all air from the cell, the water reservoir tank was manually elevated, the outlet valve opened, and the cell water inlet valve opened until air-free water flowed from the air escape valve. The two water valves were then closed and the reservoir tank returned to its support. With the cell full of water, the air escape valve and the bypass valve were closed. The cell water inlet valve and the valves to the mercury reservoir and the measuring tube were opened.

Loading Procedure for Routine Tests

With the completion of the above steps, the specimen was ready to test. An initial reading of the level of the mercury in the measuring tube and a deformation dial reading were obtained. The specimen drainage valve was adjusted in accordance to the type of test, closed for the "Quick" test, and open for all other types of tests. The desired confining pressure was applied to the water reservoir with adjustment of the pressure accomplished through the control valve on the nitrogen tank. To start the test, the water reservoir outlet valve was opened permitting the test pressure to be applied to the system while simultaneously a weight was placed on the 10:1 load hanger to balance the cell pressure acting on the loading piston.

Regardless of the type of test being performed, readings of the volume measuring tube and deformation dial were obtained at 15 seconds after the application of the confining pressure. The 15-second reading was utilized during the computation stage as will be discussed under that heading.

In the performance of a "Quick" or "Q" test, a reading of both the volume measuring device and deformation dial was obtained at 5 minutes after the application of the confining pressure. At this time the first axial load was applied. Readings of the volume-measuring device and deformation dial were taken at 5-minute intervals just prior to the application of the next axial load increment. In the interval between applications of axial loads the leveling device was manipulated to maintain the hanger loading bar as close to horizontal as possible. Axial loads were applied in this fashion until failure of the sample occurred. Failure was considered to have taken place when (a) axial deformation exceeded one centimeter, (b) the leveling adjustment could not maintain pace with deformation and/or (c) the amount of volume change was so great as to present a danger of forcing the mercury level beyond the limits of the volume measuring device.

If "Consolidated Quick" or "Consolidated-Partially Drained" tests were being performed, readings were made of the volume measuring device and the deformation dial at times of 0.25, 0.5, 1, 2, 4, 8, 15, 30, and 60 minutes after the application of the confining pressure. A plot of the volume measuring device readings versus time (to a logarithmic scale) was prepared as the readings were observed. Full compaction of the specimen under the applied confining pressure was assumed to have occurred if the 60 minute reading plotted as a straight line with the preceding points at a slope appreciably less than that defined by the plotted points at earlier times. This process is similar to that used for "time curves" for various load increments in consolidation tests of soils.

For "Consolidated Quick" or "Q_c" tests, the specimen drainage valve was closed at the 60-minute reading after the application of the lateral pressure. Axial load increments were applied and readings observed in the same way and at the same rate as for the previously described "Quick" tests. For "Consolidated-Partially Drained" or "CP" tests, the tests were completed in exactly the same fashion as for the "Q_c" tests except that the specimen drainage valve remained open throughout the test.

Tests at the three drainage conditions described were performed at confining pressures of 15, 30, and 60 psi on specimens made at 4 and 5 percent asphalt contents by weight of mixture and 150 and 400 psi compaction pressures. "Quick" tests at the same three confining pressures were performed on specimens prepared at 3, 4, and 5 percent asphalt contents by weight of mixture with compaction pressures of 250 and 500 psi. Additional "Quick" tests were performed on specimens fabricated at 150 and 400 psi compaction pressures at an asphalt content of 3 percent by weight of mixture and using the specified confining pressures. Typical data and computation sheets are presented in Appendix B.

Slow Rate of Loading Tests

Three tests were performed at a confining pressure of 30 psi and using an interval of one hour between the application of axial load increments rather than the 5-minute interval used for the routine tests.

The specimens were prepared in the usual fashion at a single asphalt content and compaction foot pressure. One specimen was tested at each of the drainage conditions represented by the "Q", "Q_c" and "CP" types of test. With the exception of the rate of axial load increment

application, the tests were performed in exactly the same manner as the routine tests.

Water Saturated Tests

Two tests were performed after water saturation of specimens which had been compacted to a given set of initial conditions of asphalt content, compaction foot pressure, and air voids.

The water saturation was accomplished by placing the specimen in an air-tight chamber and subjecting it to a vacuum for a period of 30 minutes. Without releasing the vacuum, water was permitted to enter the chamber and cover the specimen. After an elapsed time of 30 minutes, the vacuum was applied again to the chamber and maintained for an additional half-hour. The application of the vacuum was discontinued and the specimen permitted to remain in the water within the evacuated atmosphere overnight or for a period of approximately 16 hours.

Upon removing the specimen from the chamber, it was measured for volume computations and weighed in a surface-dry condition to establish the amount of water that had been absorbed.

Testing was accomplished using a "Q" test at confining pressures of 15 and 60 psi. The cell was modified for these tests to insure no drainage by introducing solid aluminium loading caps at the top and bottom of the specimen. The solid loading caps replaced ones which contained grooves to facilitate drainage and were provided with drainage outlets. The solid loading caps were used only for the water saturated tests. In all other respects the testing procedure was similar to that previously described for the "Q" test.

Disassembly of Test Apparatus

At the completion of the test, the mercury reservoir and volume measuring tube valves were closed and the bypass valve opened. Pressure was released from the water reservoir tank, the cell air-escape valve was opened, and the cell liquid drained into the water reservoir tank. The deformation dial was moved clear of the loading crossbar and all load increments removed from the hangers. The cell water-inlet valve was closed before the cell was completely drained to prevent air from entering the plastic tubing system. The volume-measuring assembly was disconnected from the triaxial cell and the water remaining in the cell recovered in a glass cylinder for return to the water reservoir tank. The cell was removed from the controlled-temperature room for disassembly.

Upon removal of the brass cylinder from around the specimen, the specimen was inspected carefully for membrane leaks that may have occurred and would invalidate results. The membrane was stripped from the specimen and the cell cleaned thoroughly in preparation for the next test.

Evaluation of Test Procedure

Three problems were paramount in the triaxial testing procedure outlined above. The first was associated with the membranes. Occasional failures occurred and were apparent by leakage of liquid from the specimen drainage valve for tests where this valve was open, by continued unusual movements upward of the mercury level of the volume measuring tube, or by inspection of the membrane and specimen at the end of the test. In cases where such failures occurred the test

results were disregarded and a new test performed. A second problem was leakage of the equipment during the test. Close observation of all seals, joints, etc. was maintained during all tests. If care was exercised in the set-up, no leaks occurred. The last problem involved the change in volume of the cell and the volume measuring equipment under the application of the confining pressures. This problem was solved by periodic calibration tests made under all test confining pressures and by using a steel specimen. The volume changes observed during calibration trials took place almost instantaneously and were recorded for a correction of the 15-second readings taken during tests. As all other test data involved the use of differences in successive readings any errors in equipment that may have been introduced in a particular test were contained within the 15-second reading. Calibration values for the confining pressures used in the reported tests are presented in Appendix D.

All equipment performed creditably. The procedures outlined above proved to be both reasonable and adequate though they were time-consuming and occasional membrane failures, leaks, etc. were discouraging.

RESULTS AND COMPUTATIONS

This section serves to summarize the observed test data, to present the computations used, and to provide a statement of the results of the tests and computations. The section is divided into sub-sections based on the major areas of the study.

Observed Data

The compacted specimens were weighed to nearest one-half gram and the diameter and height determined as an average of at least three measurements. All linear measurements were made to the nearest one-hundredth of a centimeter. These measurements provided the initial height and volume values of the specimen upon which all computations were based.

The weights of aggregate and asphalt recorded for each mixing operation permitted the computation of the average asphalt content by weight of mixture for each specimen. The measured weight, volume and average asphalt content of a specimen, together with the average bulk specific gravity of the aggregate blend, the specific gravity of the asphalt, and the percent absorption of asphalt by the aggregate made it possible to compute the volumes of aggregate, asphalt, and air in the specimen.

During the process of performing each triaxial test, readings were taken of the extensometer used to measure the change in height of the specimen and of the level of the mercury in the volume measuring tube

at predetermined intervals. These readings permitted the axial strain to be computed for each load increment and provided a means of establishing the change in volume of the specimen under the various loads.

A typical mix record, initial characteristics computation, and test data sheets and computations are shown in Appendix B.

Stress-Strain Relationships

Loads were applied to the test specimen in multiples of a nominal 10 psi to a four inch diameter specimen. A correction to the applied load was made for the friction developed between the piston and the cell. Thus, the total load applied to the specimen was known for each increment of load.

Preliminary tests indicated that volume changes occurred under load to such a magnitude that adjustment of the specimen area was required for the computation of applied stress. This adjustment was made on the assumption that even though the specimen changed volume it would retain a uniform cylindrical shape. This is not strictly the case; however, it is felt that this assumption provided the most reasonable method of computing the average stress acting on the specimen under any given load increment. The average areas were computed in the following fashion:

(1) The initial volume of the specimen was adjusted in accordance to the volume change that occurred under any load as indicated by the volume measuring device.

(2) The height of the specimen under the same load was obtained by subtracting the change in height from the initial height of the specimen.

(3) The average cross-sectional area of the specimen was computed as the quotient of the specimen's volume divided by its height.

Axial strains were determined by dividing the change in height of the specimen by its initial height. In some cases there were slight upward movements of the extensometer during the period of application of the confining pressure only. Strain computations ignored these movements and were based on a zero dial reading obtained at the start of the test.

Typical stress-strain curves are illustrated in Figure 12 for one of the routine test series. Figure 13 shows similar data for one of the three slow rate of loading tests. All tests resulted in curves of a similar shape. In particular, the curves followed a straight line from zero deviator stress to a point designated as the Proportional Limit (P.L.). Beyond the P.L. the stress-strain curve deviated from the straight line at a rate that increased with axial strain. In approximately one-half of the tests, a peak stress value was obtained which was followed by a decrease in stress with further strain. In tests having this characteristic, pronounced shear planes were observed in the specimen. In the remainder of the tests a bulging of the specimen was noted and a typical plastic failure stress-strain curve resulted. Figure 14 shows a typical specimen before testing and representative failures. The failure planes shown in this figure have been emphasized in order to be visible in the photograph.

All tests were stopped at approximately five percent strain. At this point, the specimens had either developed a shear failure or the

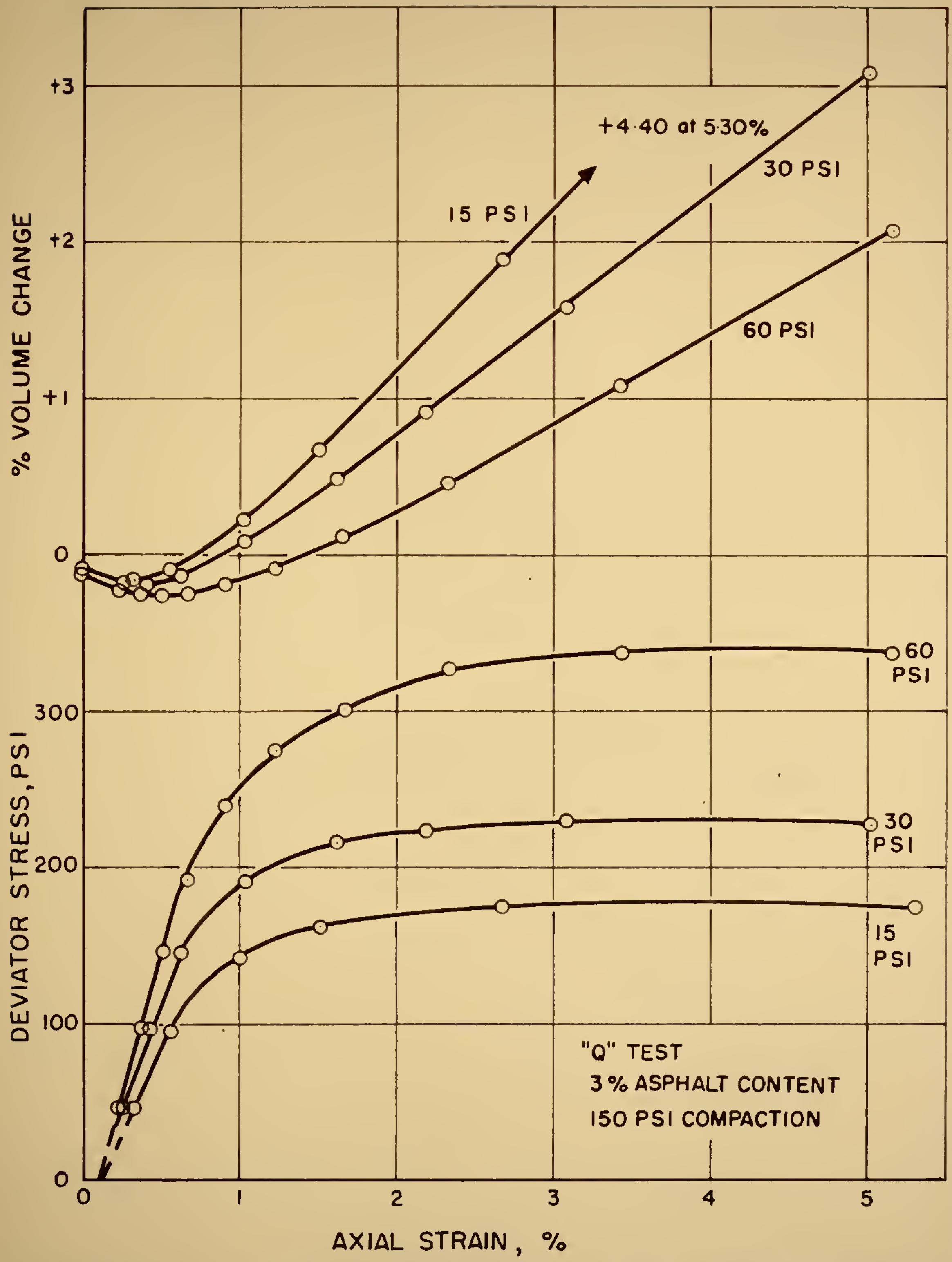


FIGURE 12. STRESS AND VOLUME CHANGE VS STRAIN

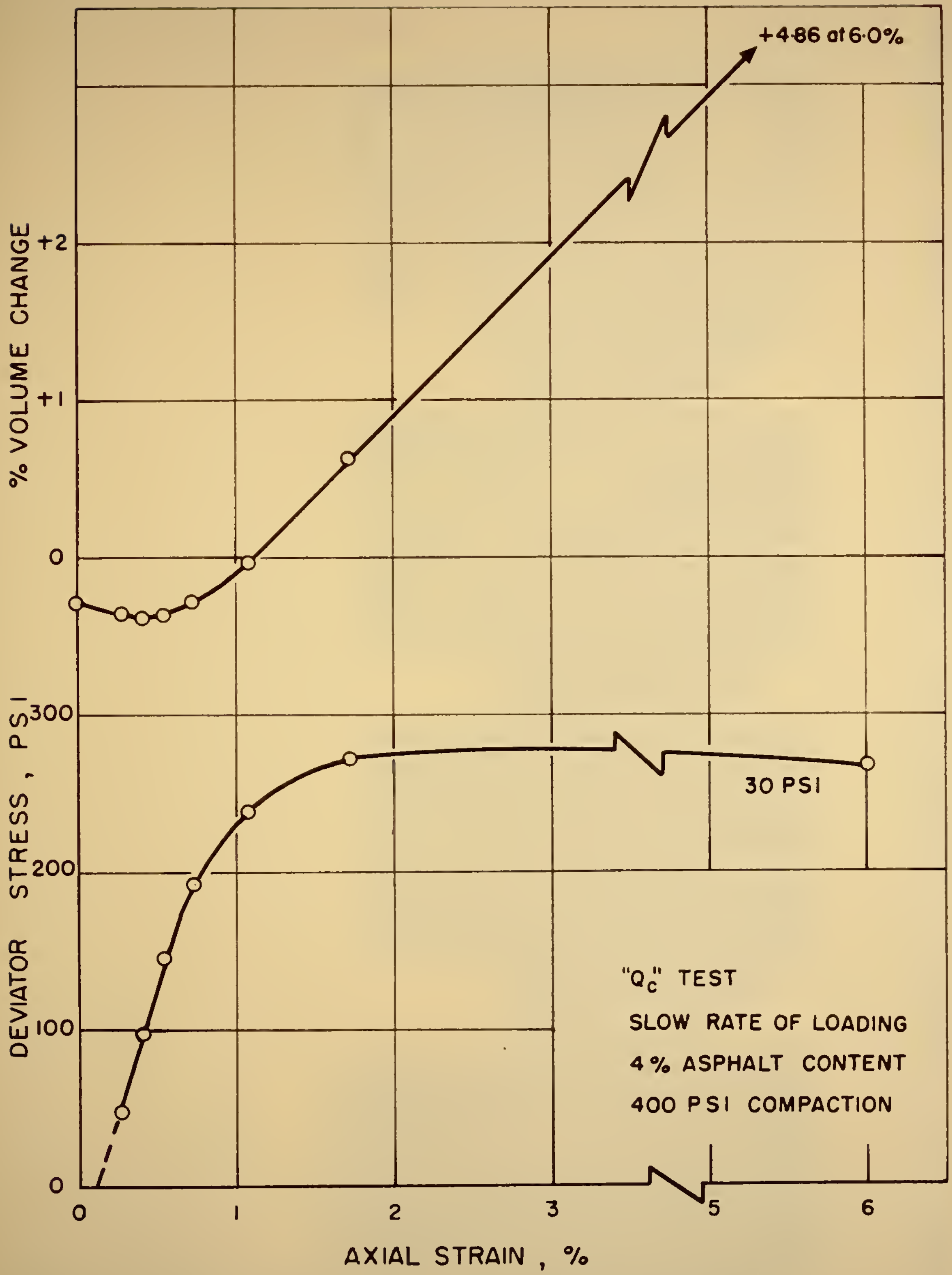


FIGURE 13. STRESS AND VOLUME CHANGE VS STRAIN – SLOW RATE OF LOADING

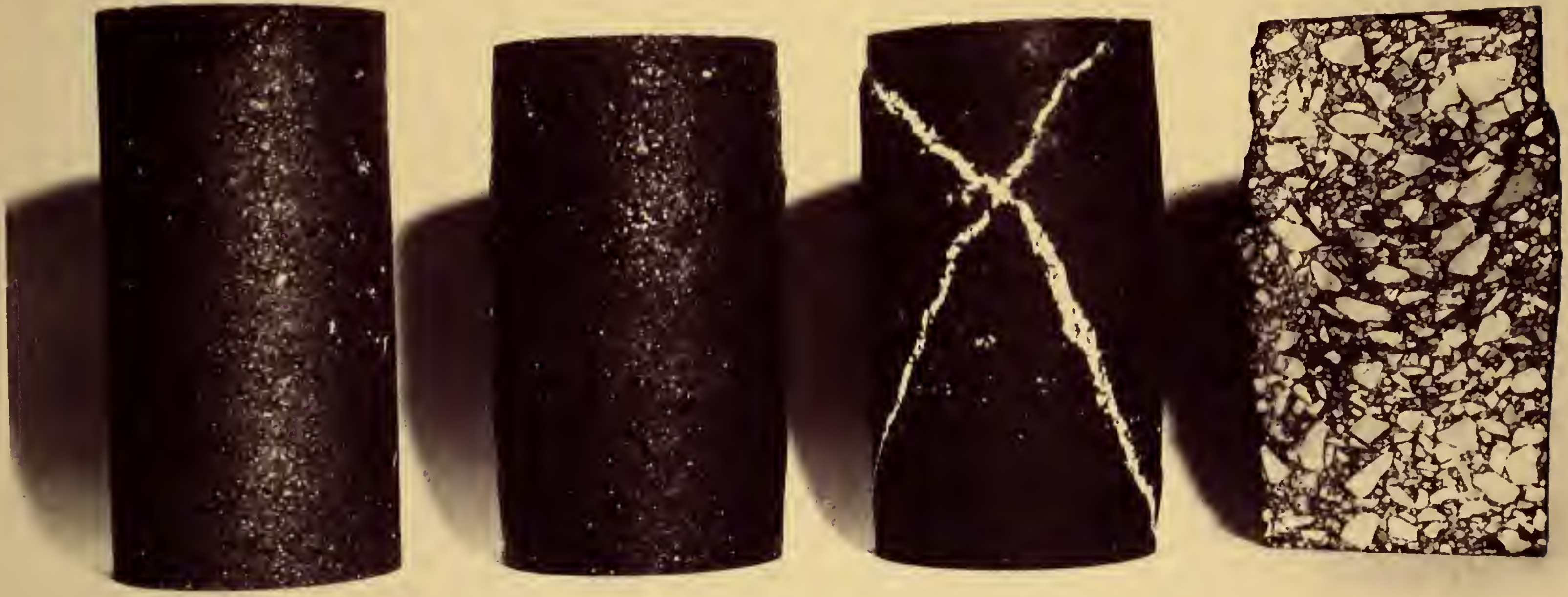


FIGURE 14 TYPICAL SPECIMENS BEFORE AND AFTER TESTING

stress-strain curve slope for bulging failures was so flat as to constitute failure for practical purposes.

Table 7 (Appendix A) presents a summary of the data obtained from all tests. It includes sufficient stress and strain values that a reasonable facsimile of the stress-strain curve for any test may be sketched.

Modulus of Deformation

The Modulus of Deformation (C_d) is defined for the purposes of this study as the deviator stress at the Proportional Limit divided by the axial strain at that point. The strain used for this computation was corrected for the intercept of the straight line portion of the stress-strain curve with the strain axis. The term, Modulus of Deformation, is utilized in accordance with common practice in the design of flexible pavements (68).

Figure 15 presents a plot of the computed C_d values for the various confining pressures for combinations of 4 and 5 percent asphalt content and 150 and 400 psi compaction foot pressures. Table 8 (Appendix A) presents the data from which Figure 15 was developed and Table 9 (Appendix A) contains a summary of the pertinent information for tests at other combinations of asphalt content and compaction pressure and for the special tests that were performed.

Strength and Stability Determinations

As stated in the Review of Literature, the question of how to establish strength and/or stability constitutes a major problem in the testing of asphaltic mixtures. For purposes of this study, deviator stresses at four points are reported: (a) at the Proportional Limit,

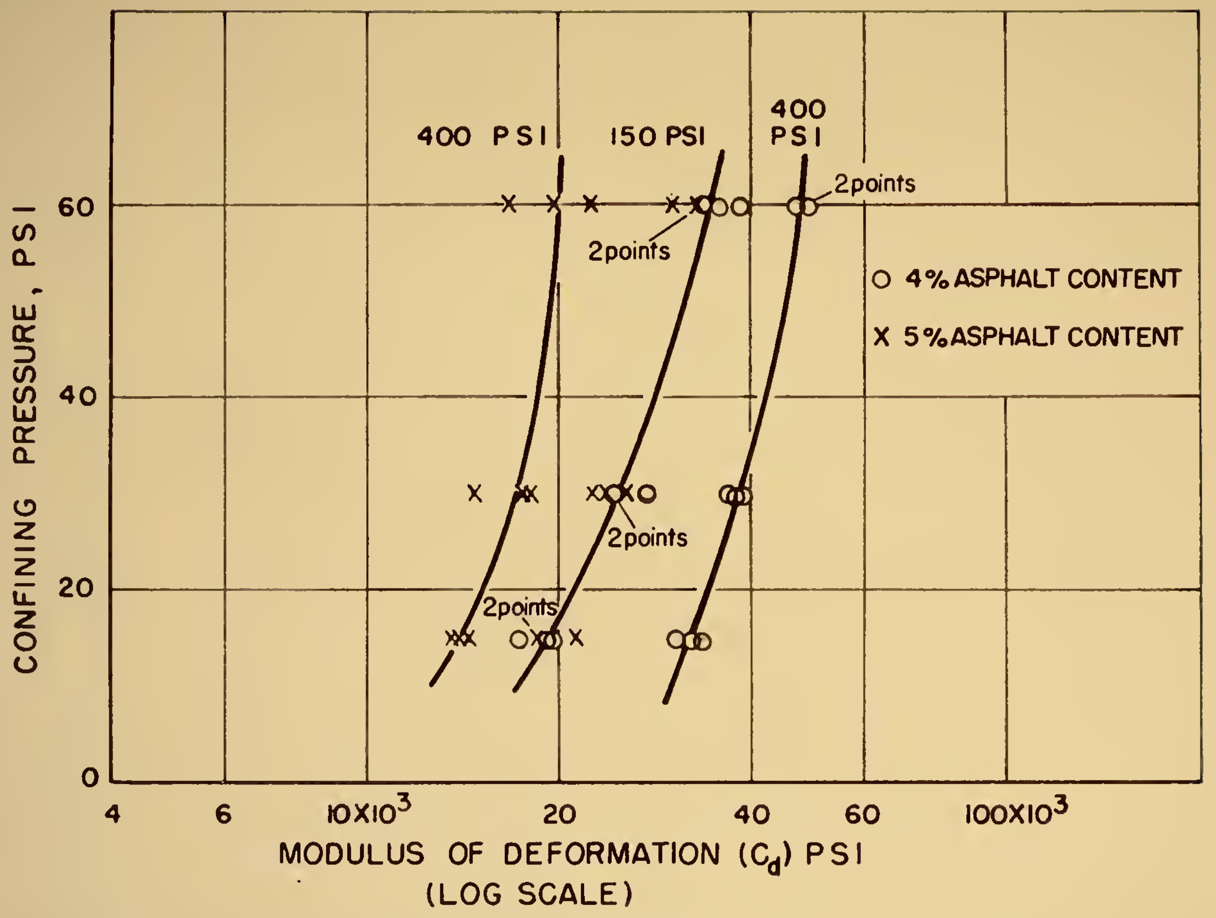


FIGURE 15. MODULUS OF DEFORMATION VS CONFINING PRESSURE

(b) at an axial strain of two percent, (c) at failure or the best estimate of the failure stress where this condition is not clearly established by the stress-strain curve, and (d) at the point where the volume change of the sample under load was zero.

The results of the strength and stability determinations are presented in Figures 16, 17, 18, 19, 20, and 21 as plots of vertical stress against lateral stress for conditions of failure, two percent strain, and at the point of zero volume change for samples prepared at combinations of 4 and 5 percent asphalt content and 150 and 400 psi compaction pressure. Deviator stress data for the same conditions for all other tests are tabulated in Table 7 (Appendix A). A typical Mohr's circle plot for one test series is shown in Figure 22 for the failure condition and in Figure 23 for stresses at two percent axial strain.

Figures 24 and 25 indicate the variation in deviator stress at failure with asphalt content for the various compaction pressures. Also shown in these figures are plots of the variation in bulk density and aggregate density with asphalt content for the different compaction pressures. Data from all routine tests are shown in these figures. For combinations of compaction pressure and asphalt content at which more than one test was performed, the average values of compressive strength, bulk density, and aggregate density are shown in Figures 24 and 25.

Figure 26 presents a graphical summary of the values of the observed shear strength parameters, "c" and "ϕ", at failure for the various asphalt contents and compaction pressures. Average values of the parameters are plotted for conditions at which more than one test series

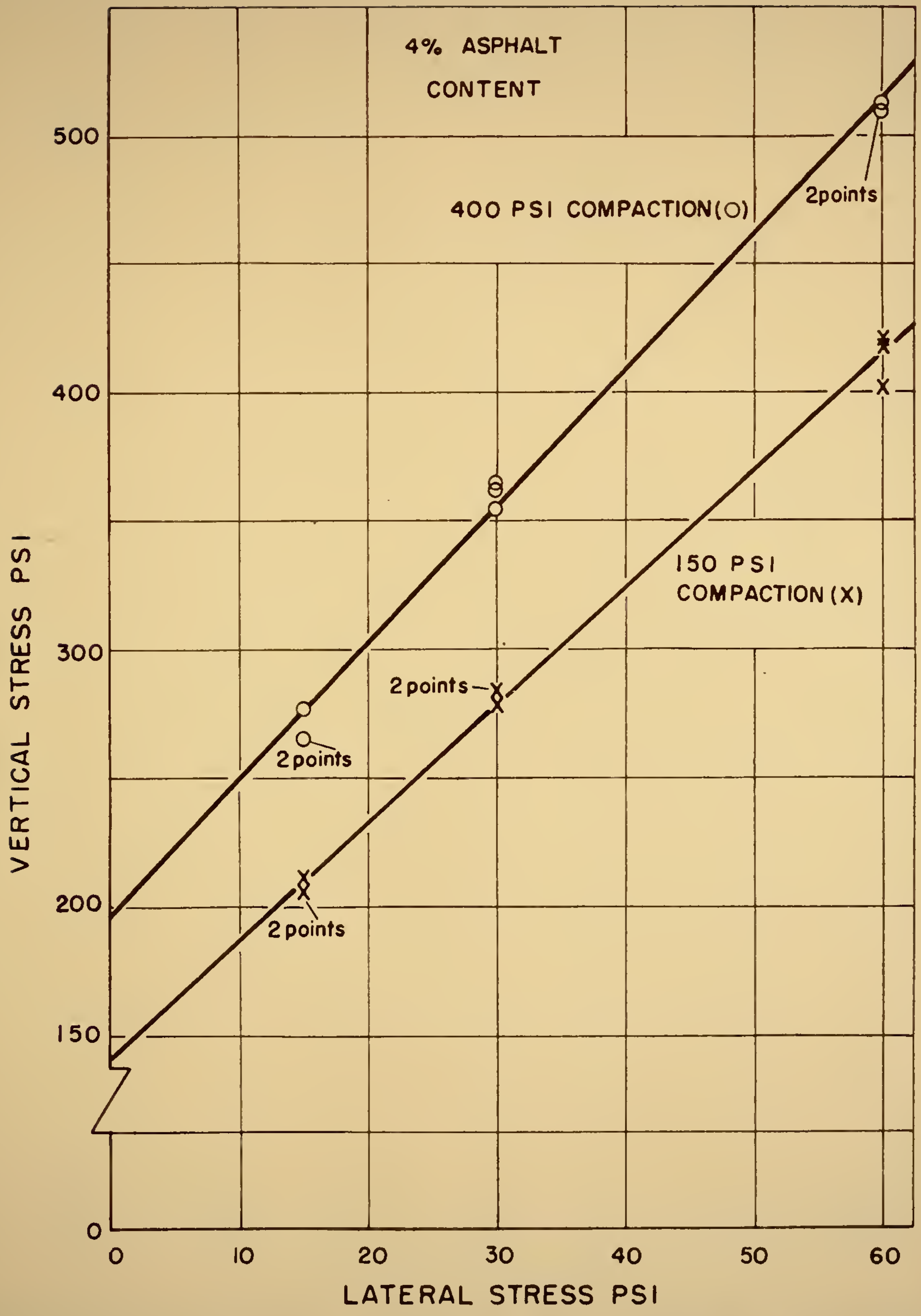


FIGURE 16. VERTICAL STRESS VS LATERAL STRESS AT FAILURE – 4% ASPHALT CONTENT

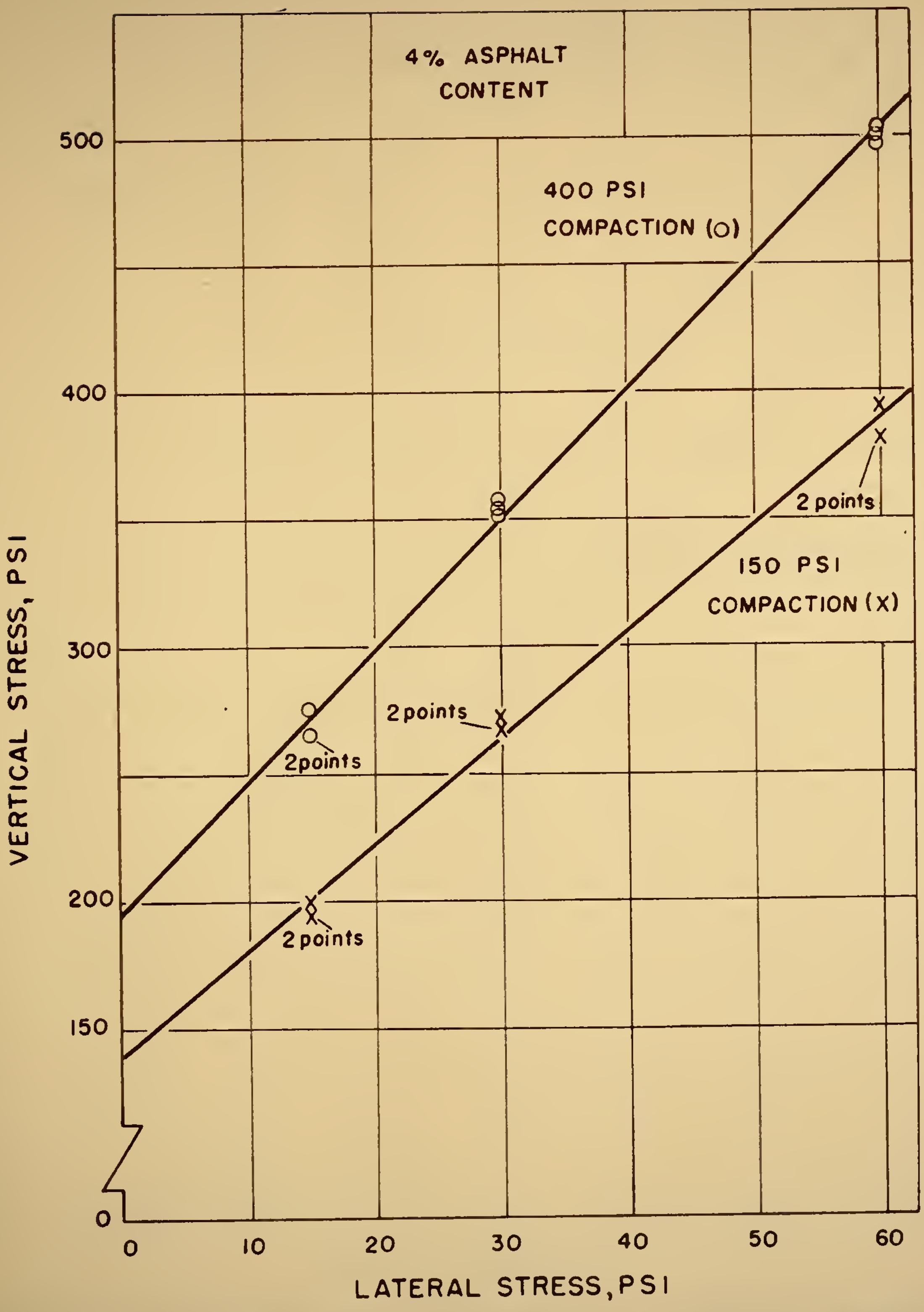


FIGURE 17. VERTICAL STRESS VS LATERAL STRESS AT 2% STRAIN – 4% ASPHALT CONTENT

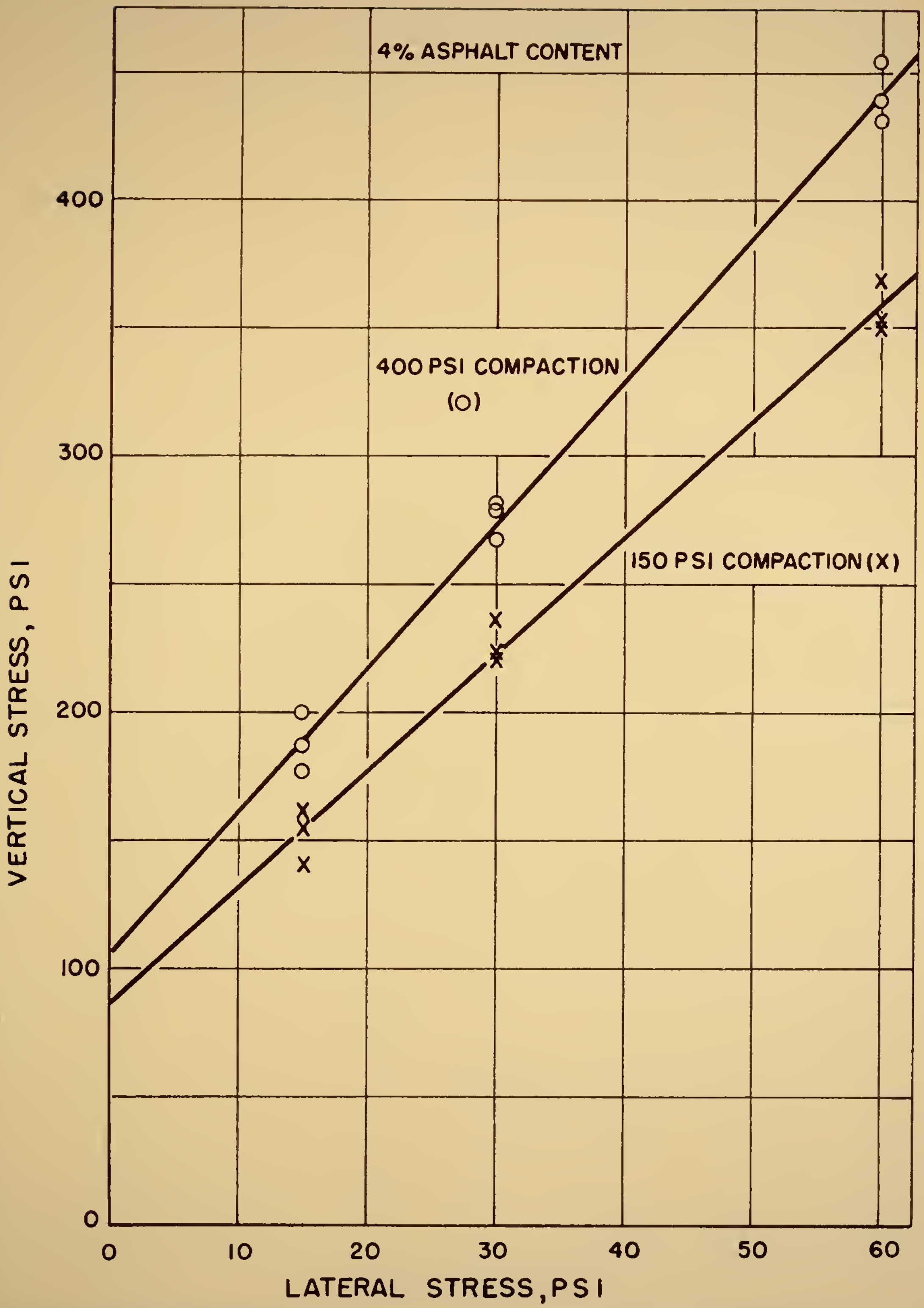


FIGURE 18. VERTICAL STRESS VS LATERAL STRESS AT 0% VOLUME CHANGE – 4% ASPHALT CONTENT

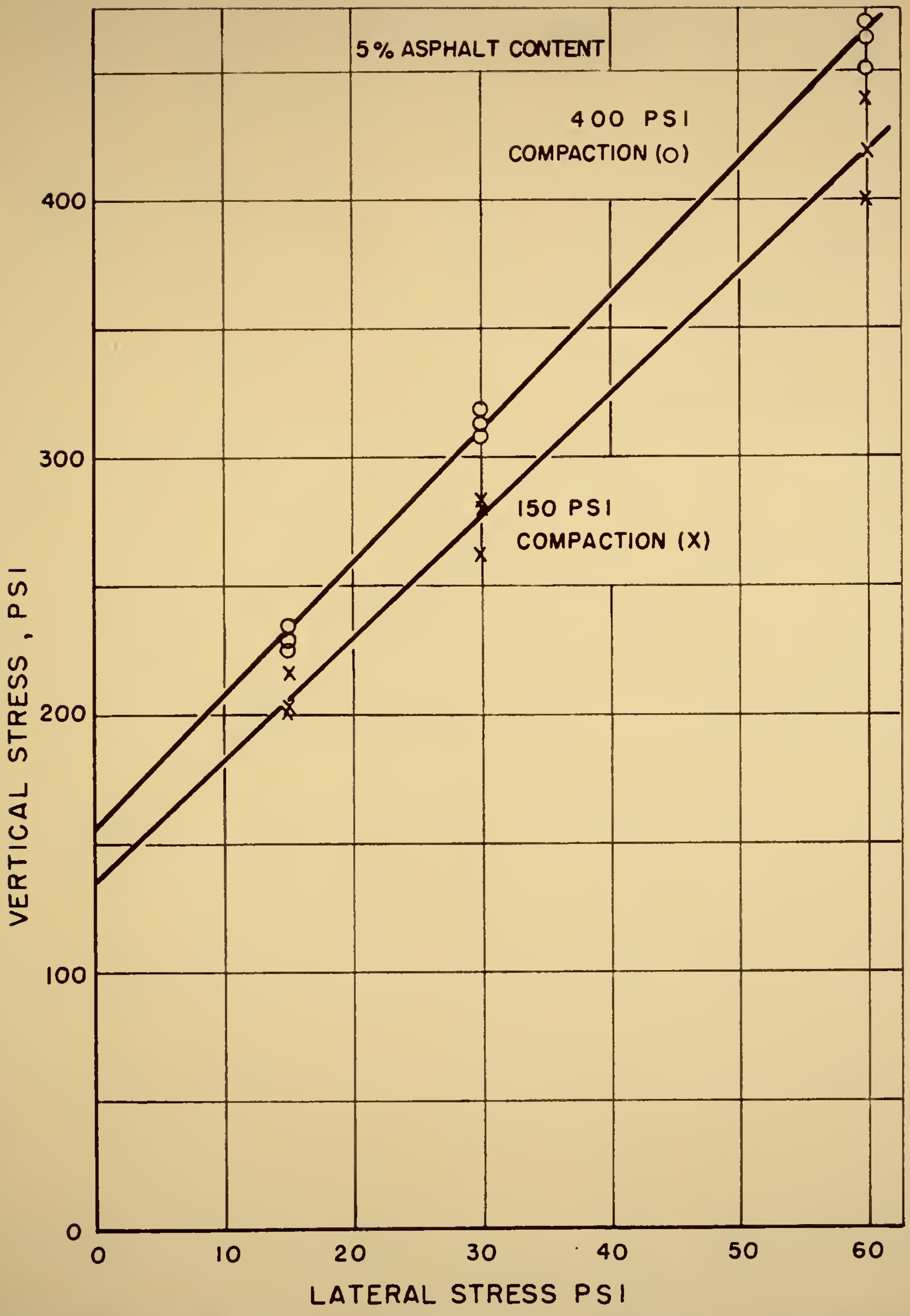


FIGURE 19. VERTICAL STRESS VS LATERAL STRESS AT FAILURE – 5% ASPHALT CONTENT

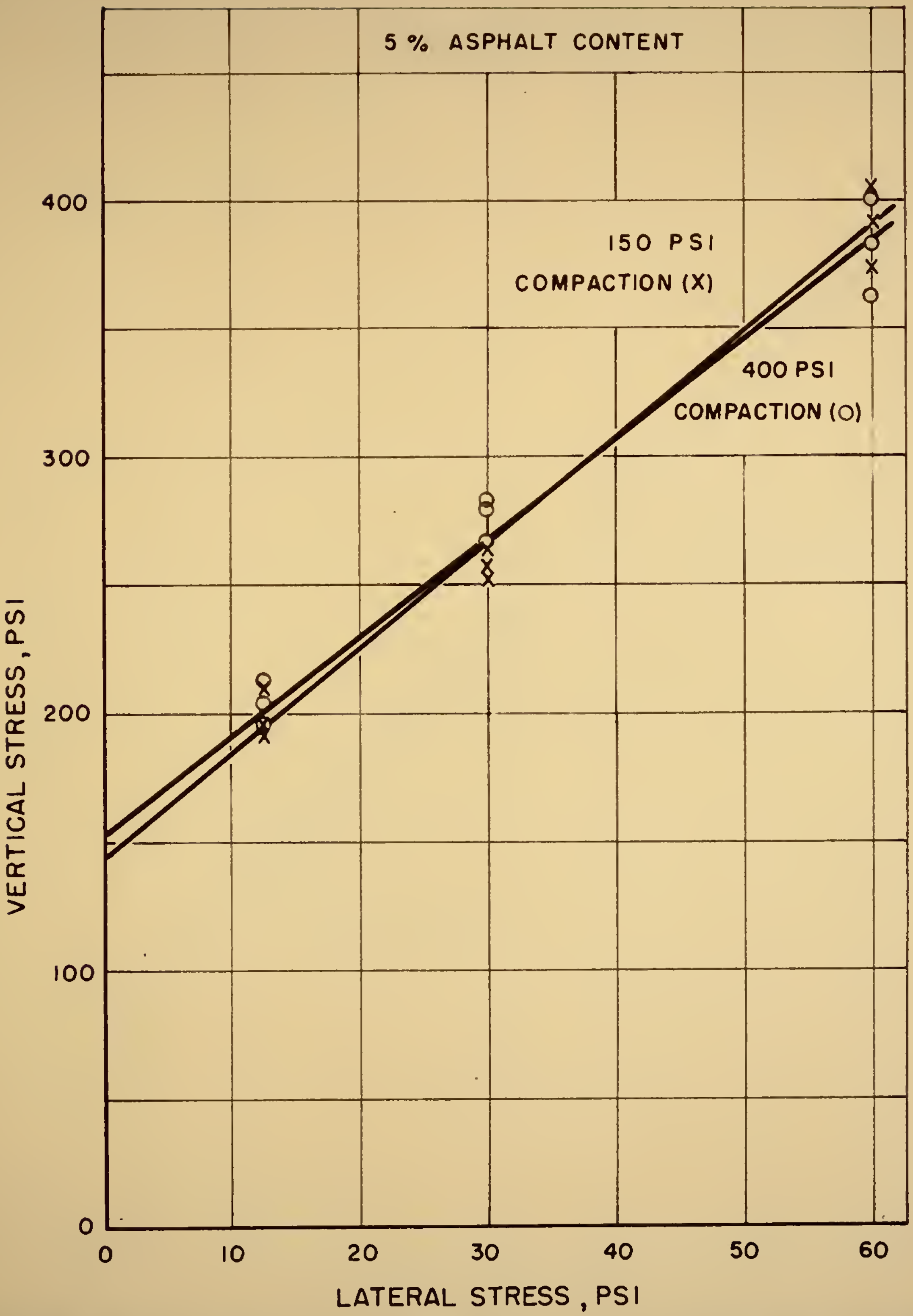


FIGURE 20. VERTICAL STRESS VS LATERAL STRESS AT 2% STRAIN - 5% ASPHALT CONTENT

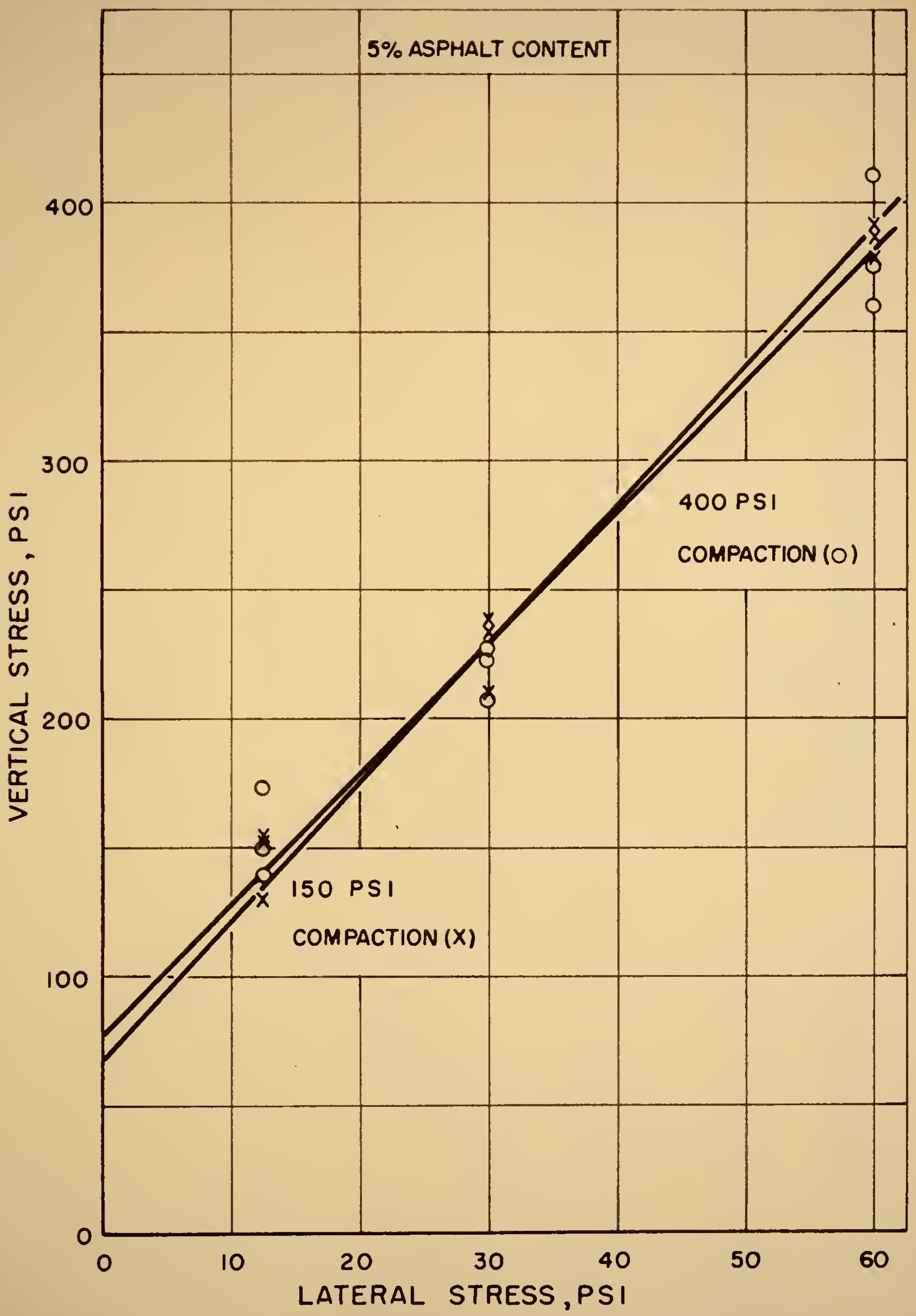


FIGURE 21. VERTICAL STRESS VS LATERAL STRESS AT 0 % VOLUME CHANGE – 5 % ASPHALT CONTENT

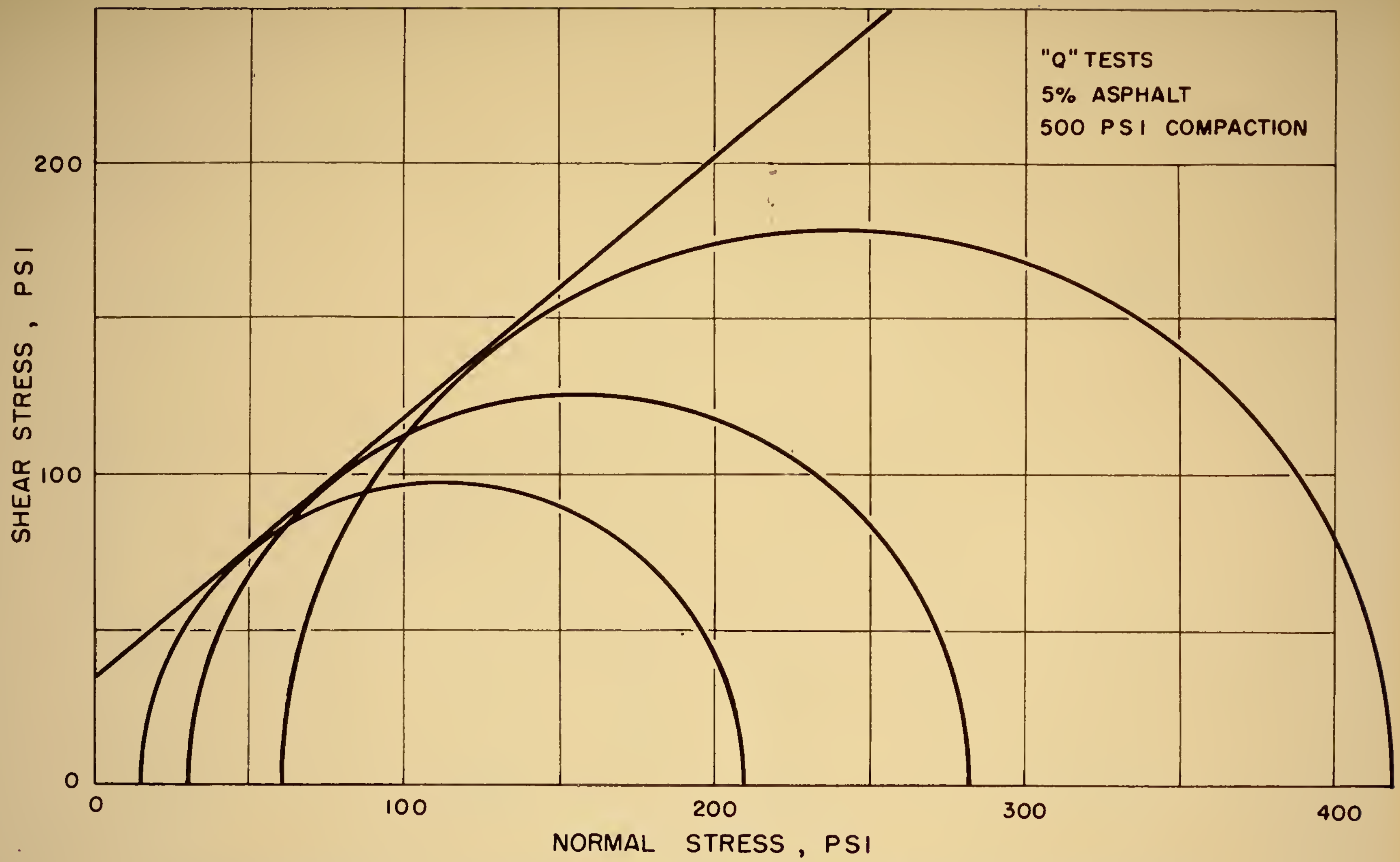


FIGURE 22. CIRCLES OF STRESS AT FAILURE

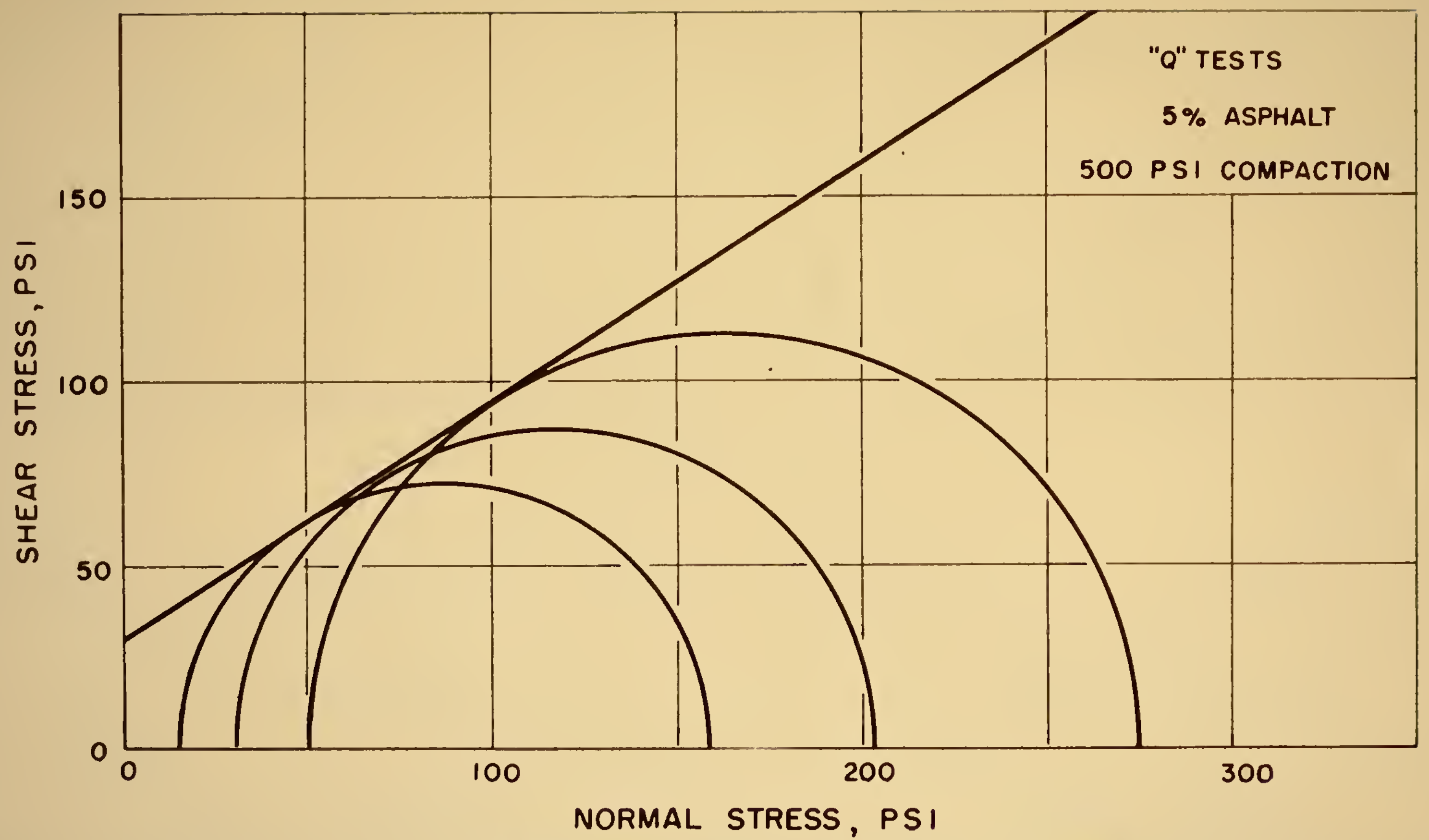


FIGURE 23. CIRCLES OF STRESS AT 2% STRAIN

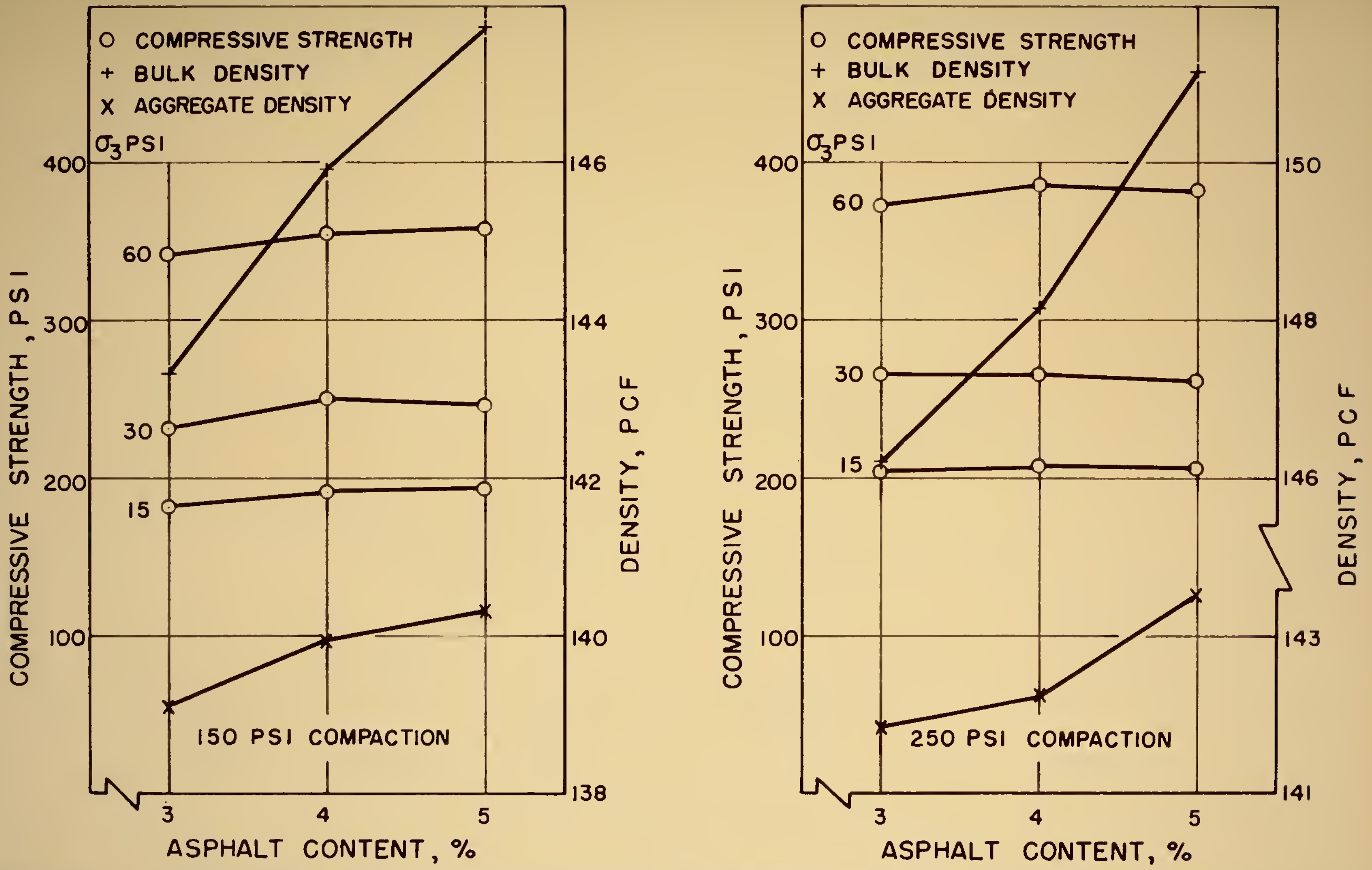


FIGURE 24. COMPRESSIVE STRENGTH AND DENSITY VS ASPHALT CONTENT — LOW COMPACTION PRESSURES

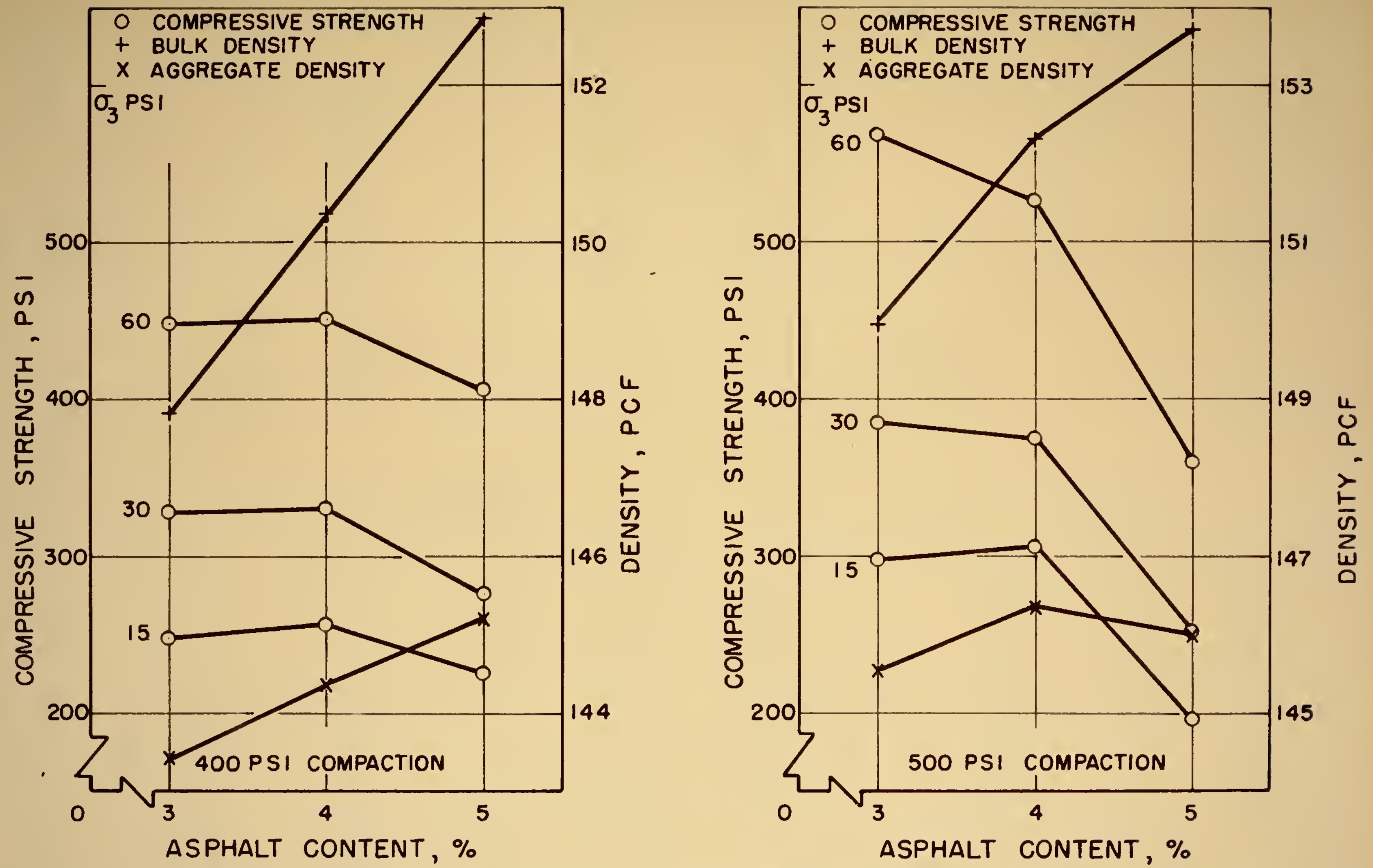
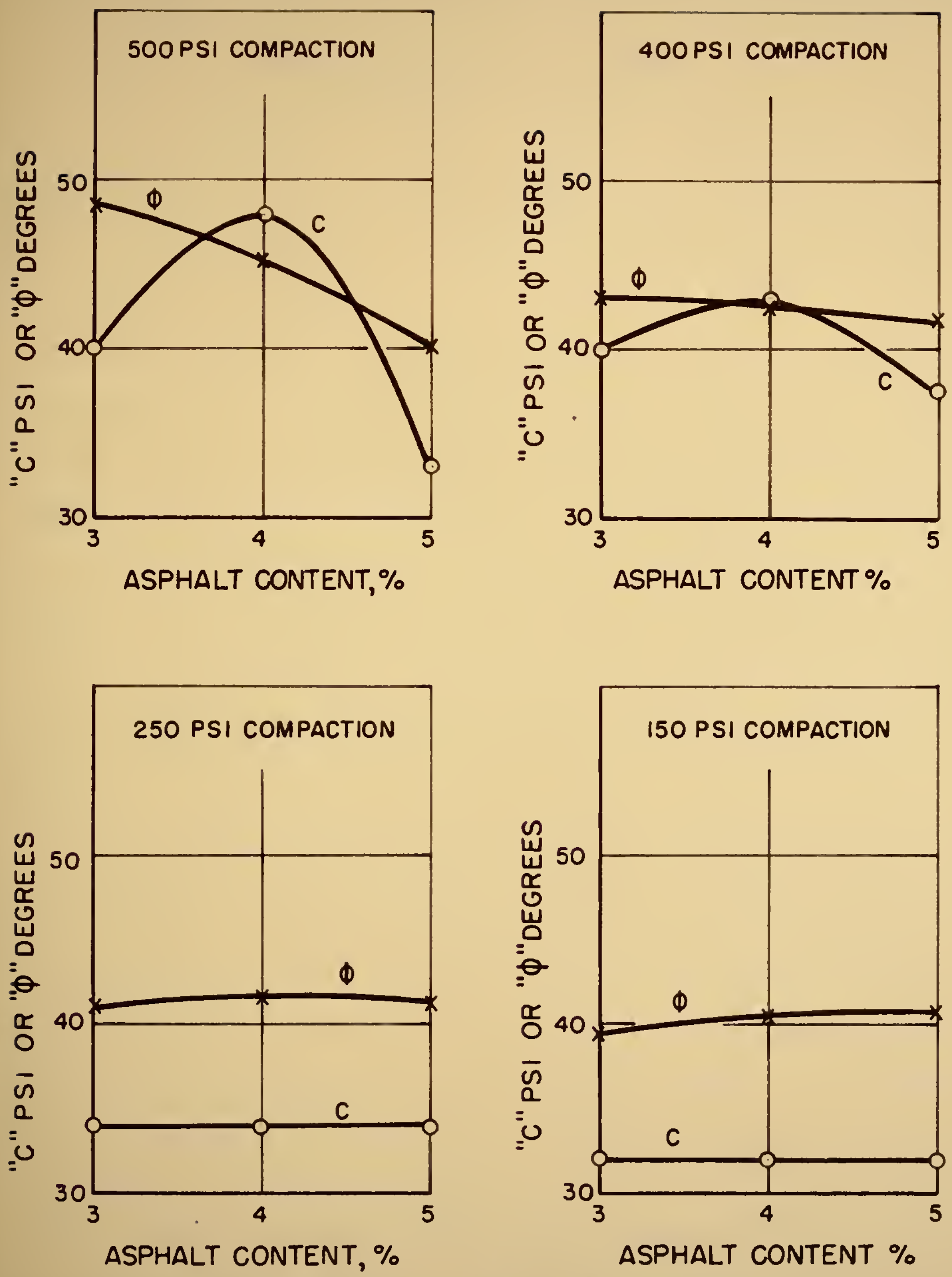


FIGURE 25. COMPRESSIVE STRENGTH AND DENSITY VS ASPHALT CONTENT – HIGH COMPACTION PRESSURES



NOTE : "C" = cohesion ; "ϕ" = angle of friction.

FIGURE 26. SHEAR STRENGTH PARAMETERS AT FAILURE VS ASPHALT CONTENT

was performed. The plotted data are tabulated in Table 10 (Appendix A).

Volume Change Characteristics

The measurement of the volume changes of the specimen under triaxial test conditions constituted a major purpose of this study. The equipment used for these measurements is discussed in the section on Materials and Procedures.

The computation of the volume of the specimen at any point during the progress of the test was based on observed changes in the volume of the cell liquid as recorded by the volume measuring device and the computation of the initial bulk volume of the specimen. An initial reading of the mercury level in the volume measuring tube provided the reference to which changes in this level could be referred for the computation of volume changes. Readings were obtained at 15 seconds after the application of the confining pressure and at periodic intervals during the testing.

It is assumed that the volume change recorded as the difference in the 15-second and initial readings is due to two separate effects. The first of these is the change in the volume of the apparatus due to the confining pressure. The second is the result of the membrane being forced into the surface irregularities of the specimen. Calibration values of the volume changes of the apparatus under the various confining pressures are shown in Appendix D.

The volume observed as the difference between the calibration volume value for a given confining pressure and the 15-second reading has been treated as a volume representing surface voids of the specimen. The magnitude of this value is directly related to the amount of the

confining pressure and the surface condition of the specimen. In general, the higher the lateral pressure, the greater the measured volume assumed to be surface voids.

The amount of volume change occurring under each successive load increment was computed from the difference in the volume measuring device readings before a load increment application and just prior to the application of the following increment. Therefore, any error induced in the computations due to air in the system, erroneous calibration, personal error in initial reading, etc., is contained within the 15-second reading. The result is, that while the absolute value of the computed volume changes may be slightly in error, the general pattern of these changes and the order of magnitude of the changes are thought to be correct.

The volume of the specimen and the volume of air within the specimen were computed by the algebraic summation of the initial values of these volumes and the incremental changes computed from the differences in the mercury levels between successive readings corrected for the volume displacement due to the movement of the piston into the cell.

Figures 12 and 13 show typical changes in volumes of specimens under the applied deviator stress as a function of axial strain. For ease in comparing the magnitudes of volume changes between specimens, the changes are presented as a percentage of the initial specimen volume less the volume assumed to represent surface voids.

All specimens showed a change in volume during the testing. In all cases there was a decrease in volume followed by an increase in volume. The increase in volume became linear with axial strain and continued in

such a manner to failure. In all cases there was an appreciable increase in volume at the failure condition. The slope of the straight line portion of the volume-change plot varied with confining pressure and with aggregate density. In general, the higher the aggregate density the greater the gradient and the higher the asphalt content the lower the volume-change gradient. An increase in confining pressure for tests on specimens having similar initial conditions caused a decrease in the gradient.

The volume changes occurring as a decrease in the volume of the specimen were greatest for the higher lateral pressures. The magnitudes of these changes in any case were small - generally less than one-half of one percent of the reference volume. The greatest decreases in volume occurred for specimens having the lowest aggregate density values.

Table 7 (Appendix A) shows the magnitudes of the observed volume changes at the Proportional Limit, at two percent strain, and at the minimum sample volume. The volume changes are reported as cubic centimeters of change from the initial bulk volume less the volume assumed to be surface voids. The percentage of the reference volume represented by this change is shown in the adjacent parenthesis.

Typical volume computations are presented in Appendix B.

Void Ratio and Percent Voids

In asphalt mixture terminology, the percent voids (%V) is defined as the amount of air volume expressed as a percentage of the total volume of the specimen. In soil mechanics terms, an expression for void ratio (e) is commonly used to define the ratio of volume not occupied

by aggregate solids to the volume of the aggregate solids. Both terms have been used in this study, the former because it provides an easy reference for those familiar with asphalt terminology and the latter because the aggregate solids volume provides an unchanging reference regardless of whether the specimen expands or contracts. A mathematical relationship between these two terms is developed in Appendix E.

Void ratio and percent voids were calculated utilizing the computed volume of the air and total specimen volume based on observed volume changes and on the known fixed volume of bitumen and aggregate solids in the specimen as computed from the initial measurements of each specimen. Aggregate volumes were computed using the average bulk specific gravity of the aggregate blend and the asphalt volume was determined from the weight of asphalt in the specimen less the weight absorbed by the aggregate. Sample computations of void ratio and percent voids are shown in Appendix B while a summary of these values at the initial condition, the Proportional Limit, and two percent strain are presented in Table 7 (Appendix A).

The change in void ratio and in percent voids during a test follows the same general pattern as that of the total volume change. Initially there is a decrease in these values and then an increase to failure. Both terms show a very gradual change in value under the intermediate applications of load. Changes are greatest under the initially applied loads and as the specimen approaches failure. Figure 27 shows the void ratio and percent voids plotted against total applied axial stress for a typical test series.

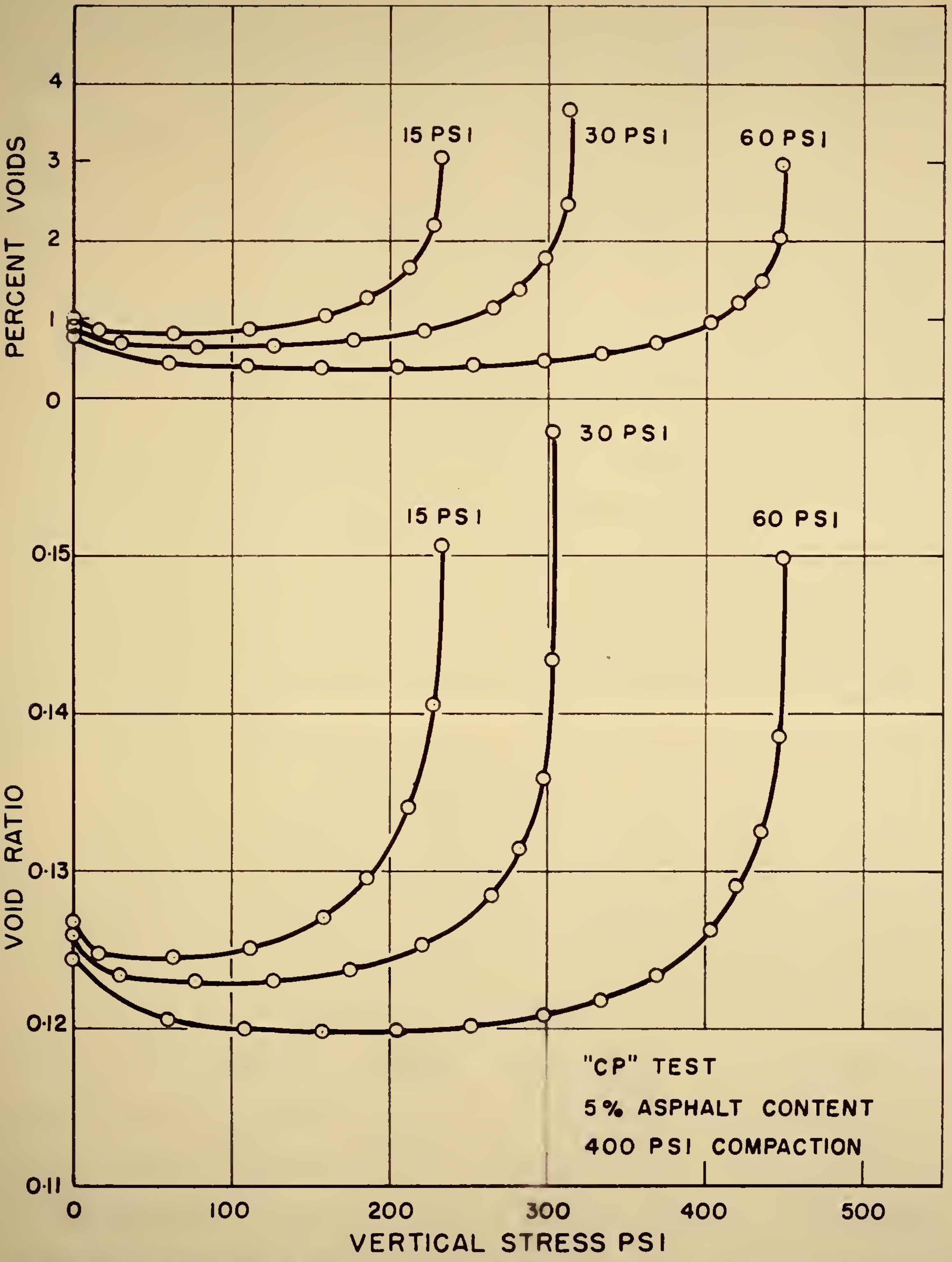


FIGURE 27. PERCENT VOIDS AND VOID RATIO VS STRESS

Figures 28, 29, 30, and 31 show the relationship between deviator stress and void ratio and percent voids at the Proportional Limit and at two percent strain for all routine tests performed on specimens prepared at 4 and 5 percent asphalt content and 150 and 400 psi compaction pressure. Similar data are tabulated in Table 7 (Appendix A) for these and all other tests.

Figures 32 and 33 show like data in a slightly different form. These figures present a plot, for all routine test results, of the deviator stress versus the percent voids at two percent strain. The curves that are approximately vertical represent the deviator stress variation with percent voids at two percent strain for a given confining pressure and compaction pressure. The percent voids data shown in Figures 29 and 31 for the two percent strain condition have been plotted also in these figures.

Deformation Ratio

The Deformation Ratio is a defined term that is applied to the ratio of lateral strain to axial strain for a material that is not subject to the strict application of Poisson's Ratio. Poisson's Ratio is defined as the ratio of lateral strain to axial strain within the elastic limit of the material concerned. As asphaltic mixtures are not strictly elastic throughout any range of deformations and as it is desirable to know the ratio of strains throughout the entire range from zero to the failure condition, the term Deformation Ratio is introduced. It should be noted that this term is analogous to Poisson's Ratio and is defined merely to extend the range of the concept expressed by Poisson's Ratio for elastic materials.

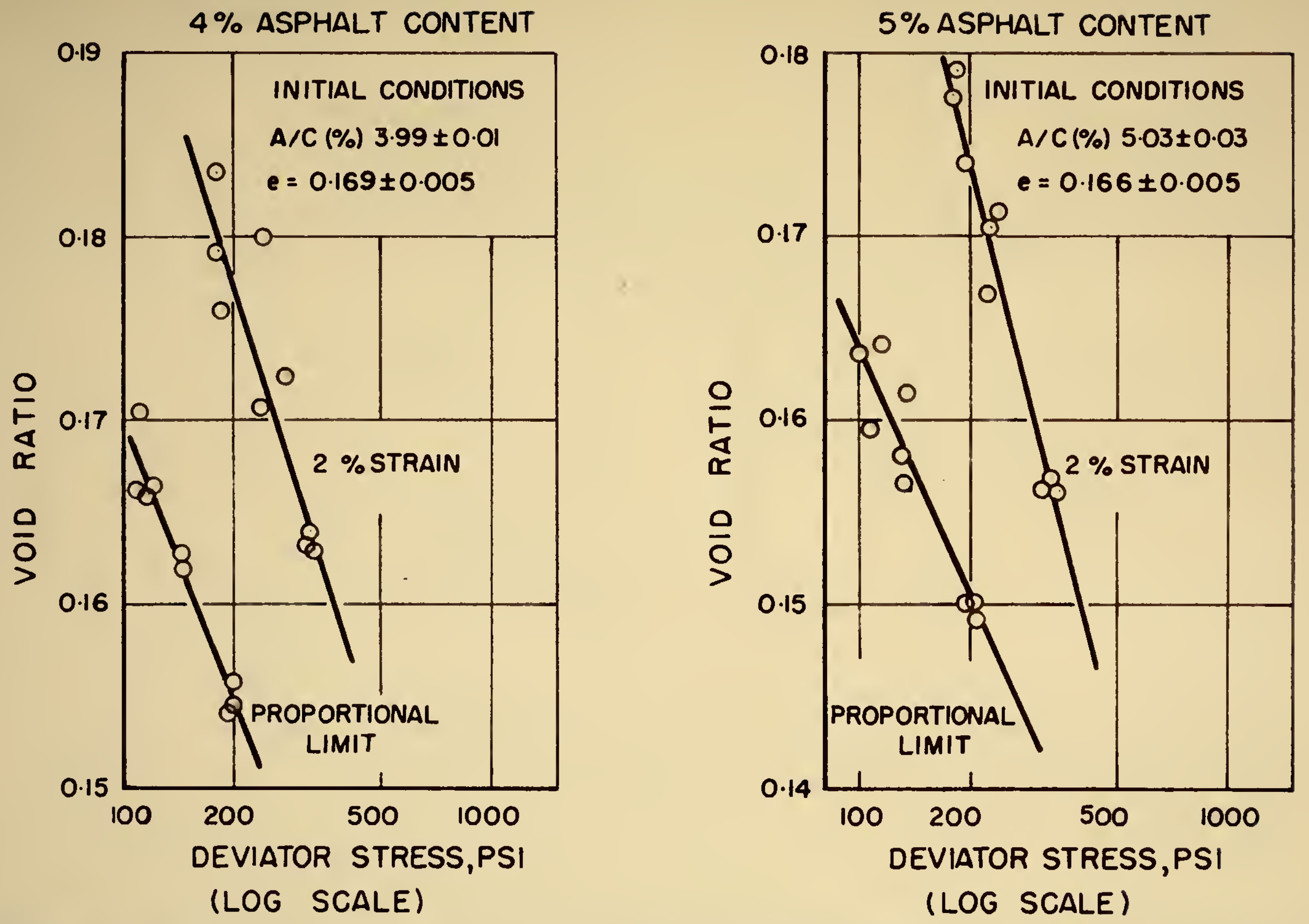


FIGURE 28. VOID RATIO VS DEVIATOR STRESS—150 PSI COMPACTION

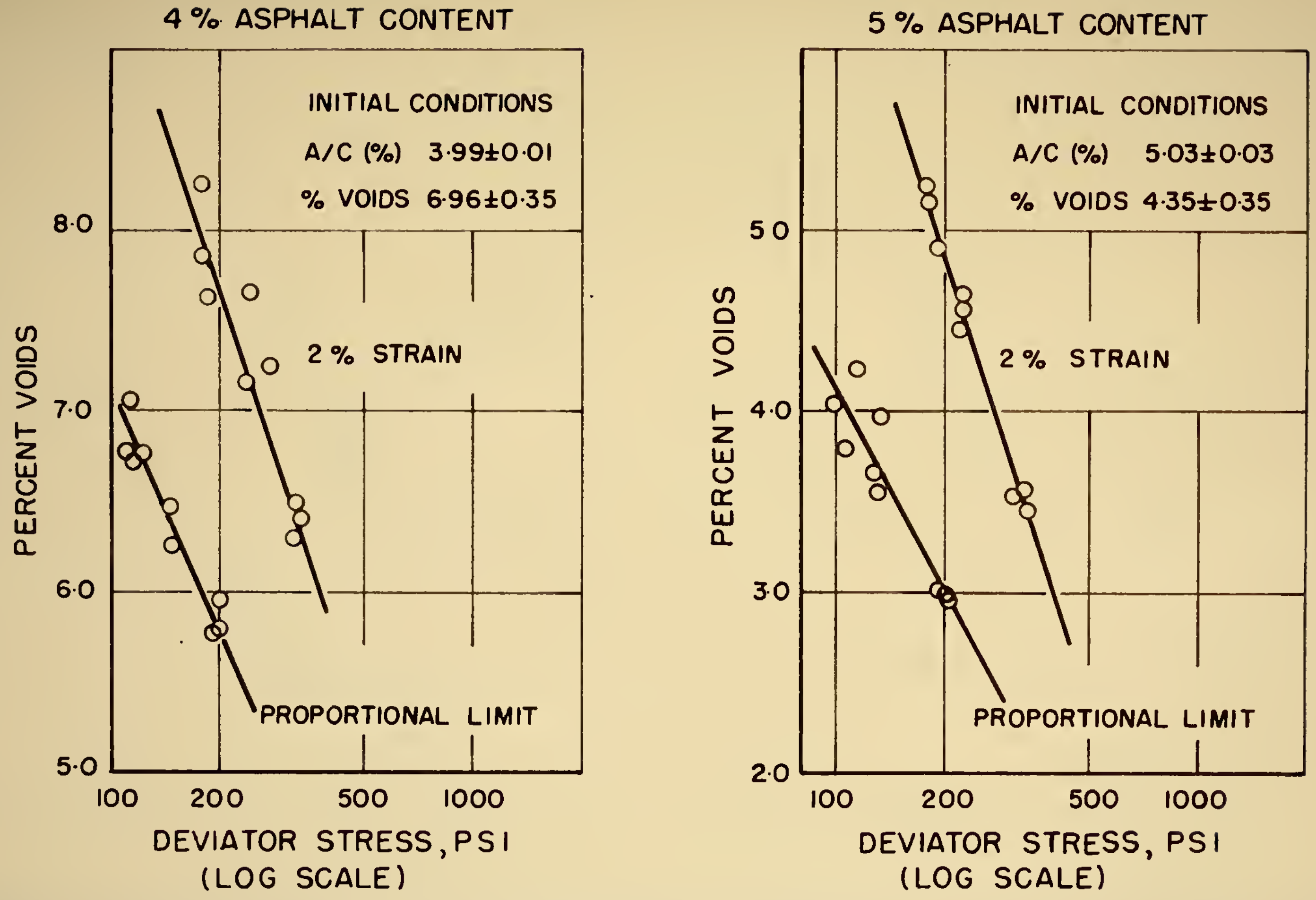


FIGURE 29. PERCENT VOIDS VS DEVIATOR STRESS – 150 PSI COMPACTION

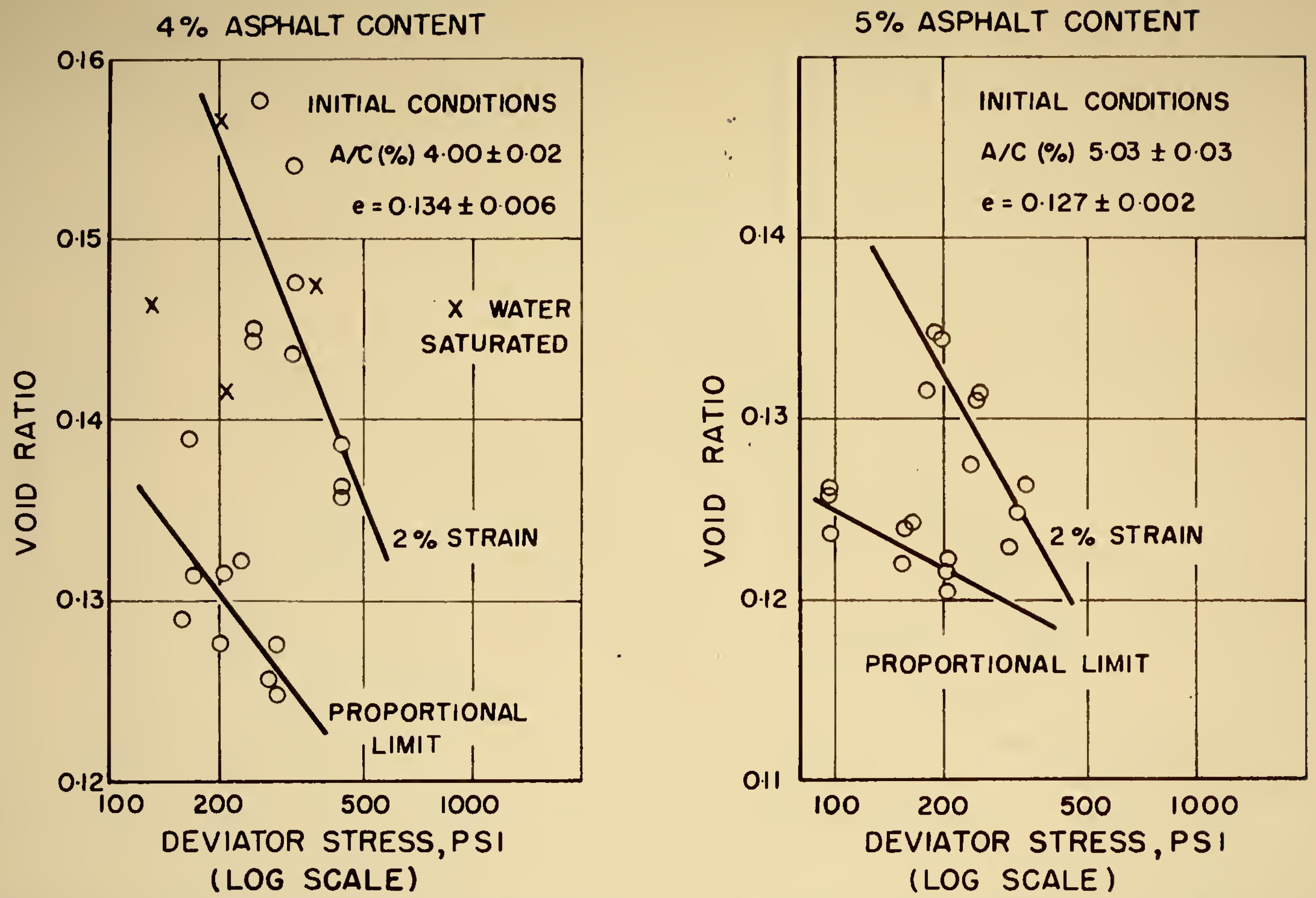


FIGURE 30. VOID RATIO VS DEVIATOR STRESS — 400 PSI COMPACTION

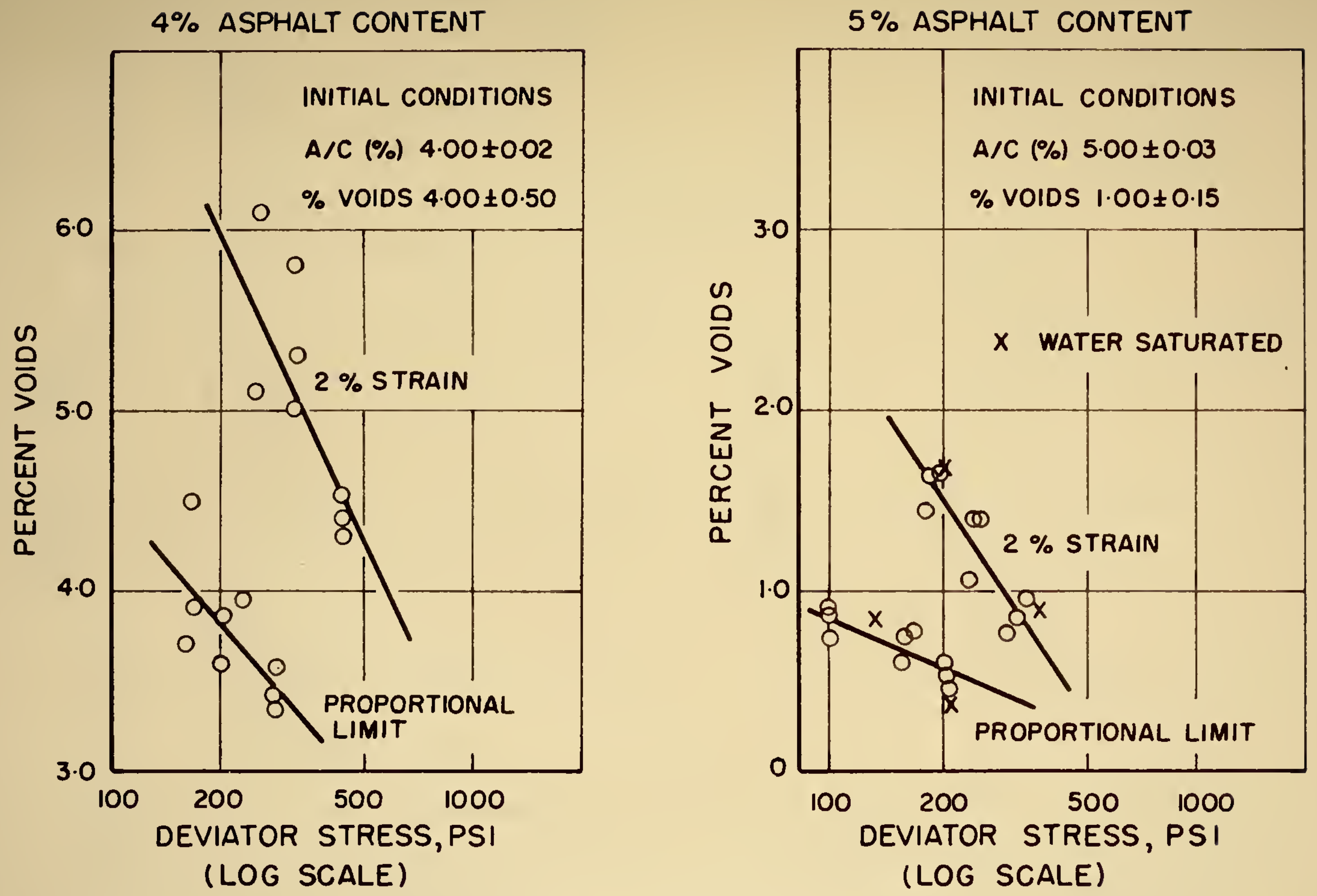


FIGURE 31. PERCENT VOIDS VS DEVIATOR STRESS — 400 PSI COMPACTION

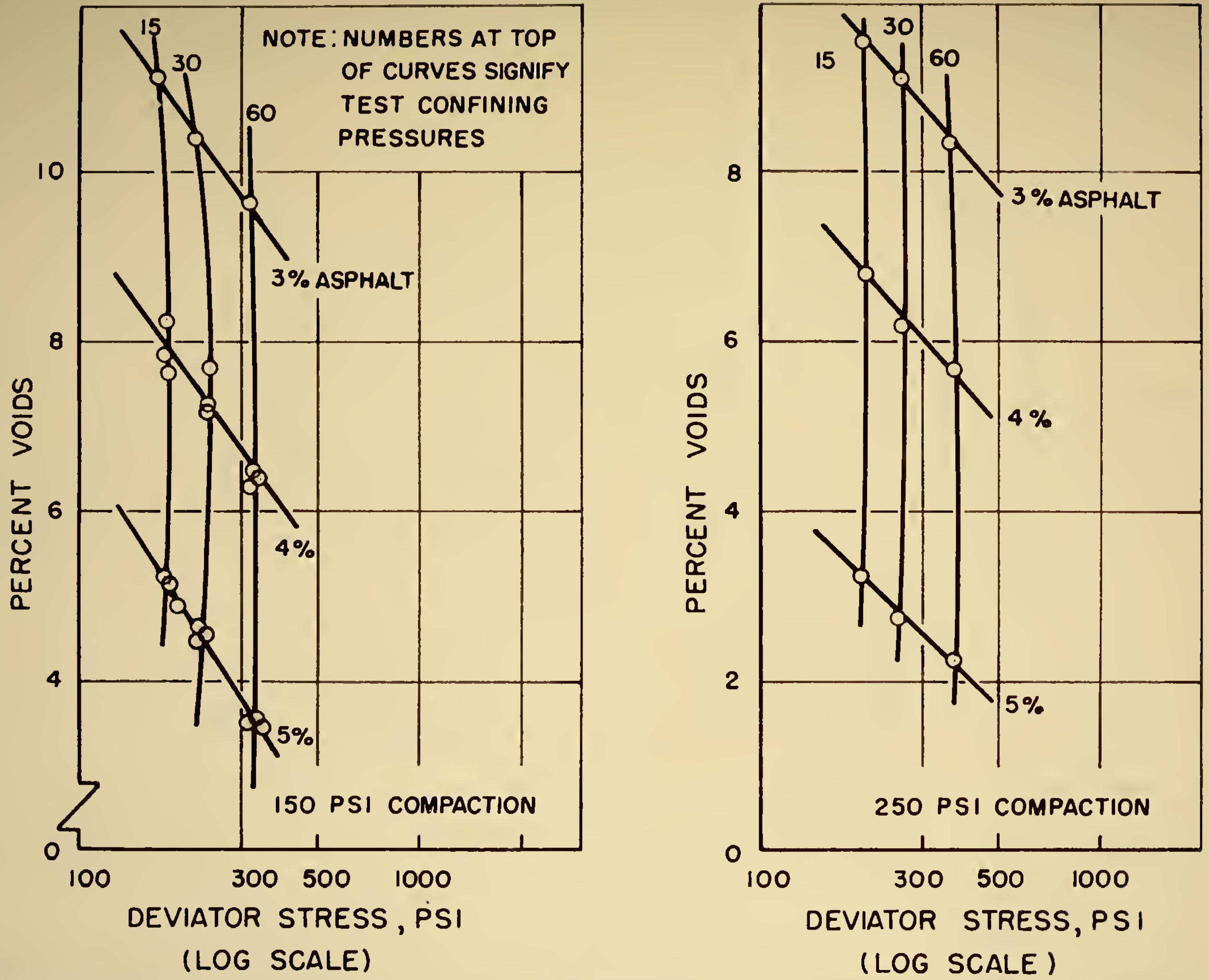


FIGURE 32. PERCENT VOIDS VS DEVIATOR STRESS AT 2% STRAIN — LOW COMPACTION PRESSURES

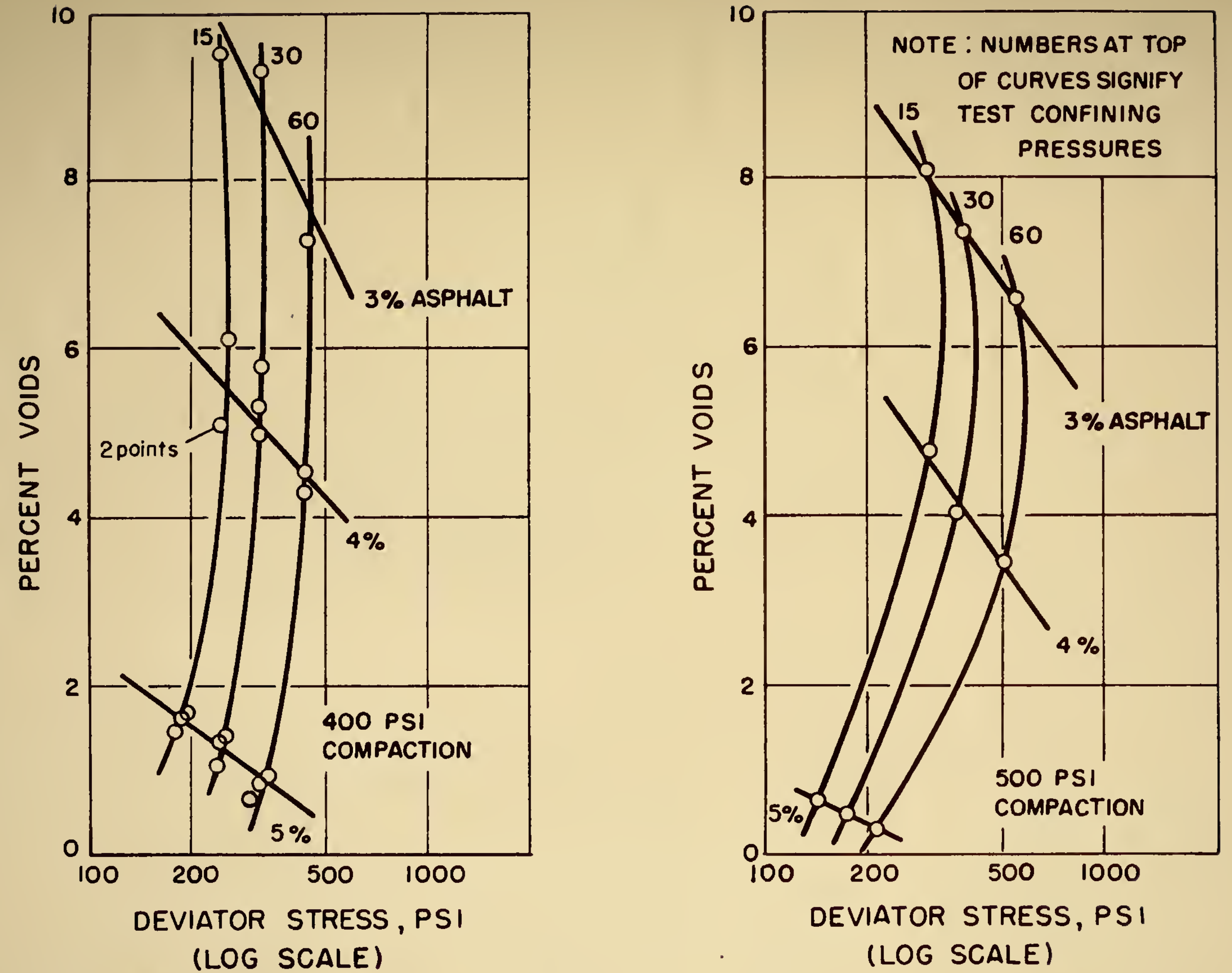


FIGURE 33. PERCENT VOIDS VS DEVIATOR STRESS AT 2% STRAIN – HIGH COMPACTION PRESSURES

Data observed during the testing permit a computation of both the axial and volumetric strains that occurred under any given load application. In the development used, these strains were referred to the specimen height and volume existing just prior to the application of the first deviator stress increment.

If it is assumed that the volume changes that occurred are uniform within the cylindrical specimen, then it is possible to develop a relationship defining the Deformation Ratio (D_r) in terms of volumetric strain and axial strain. This relationship may be stated $D_r = -(\epsilon_v - \epsilon_a)/2\epsilon_a$, where ϵ_v and ϵ_a are volumetric and axial strain respectively. The mathematics of this development are presented in Appendix E.

The D_r values have been computed for four series of "Q" tests, each at a different combination of asphalt content and compaction foot pressure. The results of these computations are presented graphically in Figure 34 for one series and in tabular form in Tables 11 and 12 (Appendix A) for the four test series investigated. In general, the presentation follows a scheme suggested by Jakobson (69).

The figure showing the results of the D_r computations presents them as plots of D_r versus sin ϕ_1, where ϕ_1 is the angle of shearing resistance that has been mobilized at that particular point in the test. The values of sin ϕ_1 were computed utilizing the fundamental relationship $\sigma_1 = \sigma_3 \tan^2(45 + \phi/2) + 2c \tan(45 + \phi/2)$ for a material possessing both friction and cohesion. To facilitate the computation it was assumed that the term $2c \tan(45 + \phi/2)$, evaluated at the failure condition, was a constant and was acting at all stages of the test. This term was

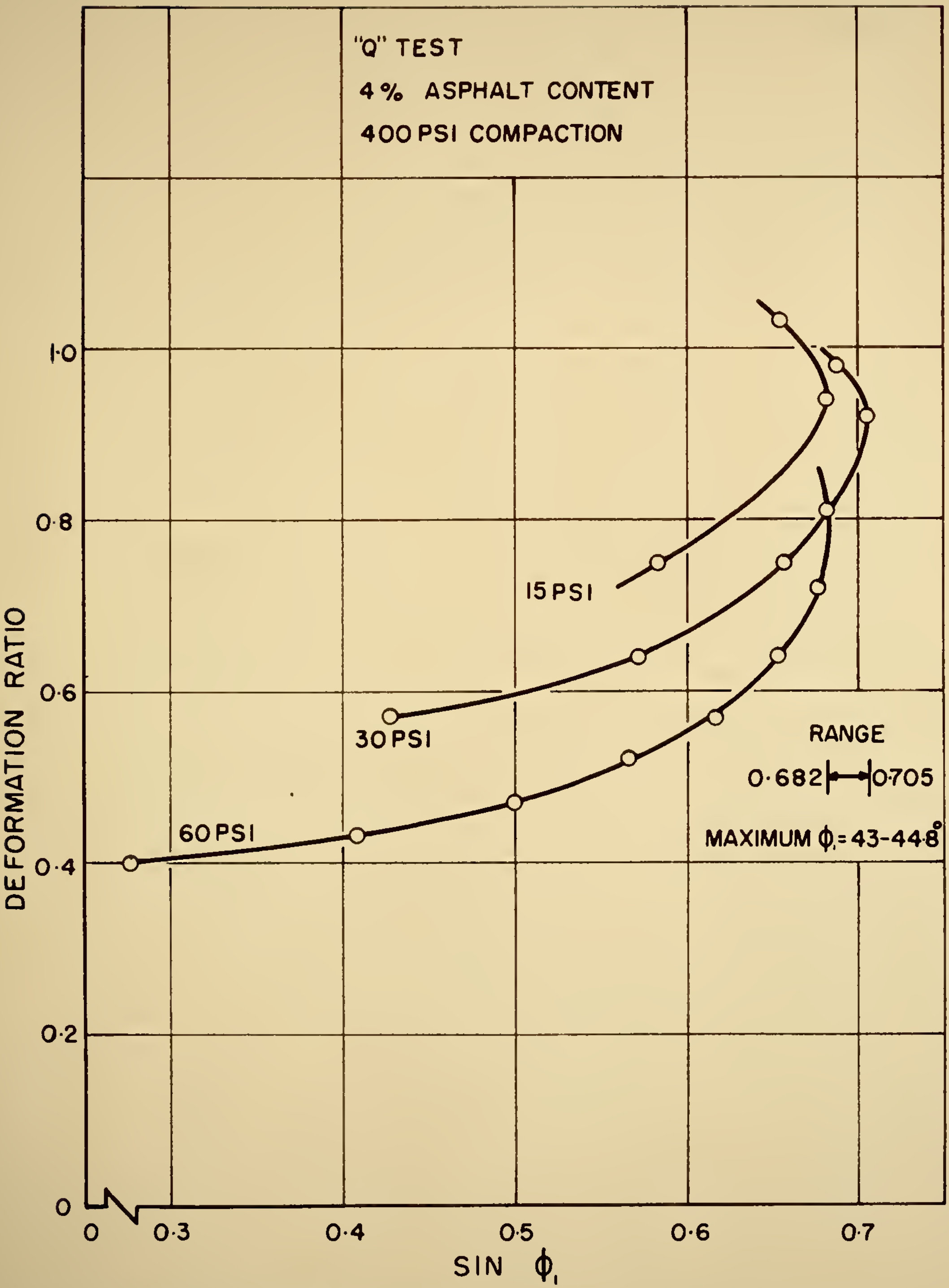


FIGURE 34. DEFORMATION RATIO VS ANGLE OF SHEARING RESISTANCE

subtracted from the major principal stress acting at each point and sin ϕ_1 computed using the frictional resistance values only. An illustration of this computation is presented in Appendix B.

Appendix B also presents a typical set of computations leading to the data required for a plot of D_r versus sin ϕ_1 for a given test. The particular computation presented is shown graphically as the σ_3 = 30 psi curve in Figure 34.

DISCUSSION OF RESULTS

The discussion of the results obtained in this study is sub-divided into eight sections. Five of these are discussions of the results obtained which have direct bearing on the purposes of the investigation, while the remaining three refer to the results that developed as an incidental part of the investigation but which are of interest in the study of bituminous mixtures.

Equipment

Two major pieces of equipment, peculiar to this study, merit discussion. The controlled temperature room was purchased in order to provide a constant temperature atmosphere in which to perform all tests. The volume measuring device was constructed to provide a means for establishing the volume changes of a specimen subjected to triaxial compression if such changes occurred.

The controlled-temperature room performed well during the entire period of usage. An established temperature of 25°C (77°F) was maintained at the sensing element within the room and in the water used for a cell liquid. Some variation in temperature was noted within the room but, in general, this variation was less than ± 0.5°C from the average air temperature at the point of measurement. The temperature records show a daily 25°C trace with a variation of the order of ± 0.5°C. The exact temperature was not so critical a part of this study as was the

constancy of the temperature during testing. The uniformity of temperature was adequate for test purposes.

Some disadvantages are inherent in the controlled temperature room. The room, as constructed, provides for a recirculation of air within the room. As a result, the room air becomes stale and odorous after a long period of use. The size of the room limits the number of persons that can work within it when the triaxial equipment is in place.

The volume measuring device performed well throughout the period of testing. Initially there was some difficulty in removing all entrapped air from the system, but this was overcome by patient bleeding and refilling of all lines and containers. The use of an epoxy resin seal on all plastic tubing joints was completely effective in eliminating leakage from these joints under the range of confining pressures developed during the testing.

Four calibration tests were performed at various times during the period of testing. The calibration test results are presented graphically in Figure 35 (Appendix D). Care was required in performing the calibration tests. Occasionally repeated applications of a confining pressure on a given calibration test set-up yielded different values. This was assumed to be due to air in the system, a poor initial reading, or other operator error. In such cases, the calibration runs were repeated until consistent results were obtained.

In summary, it may be stated that the volume measuring device performed well for the measurement of differences in volume under successive loads but some difficulty was experienced in establishing the initial volume change under the application of the lateral pressure. As all

computations were based on differences in readings of the volume measuring device, it is felt that the results obtained using the apparatus are correct as to trend and order of magnitude but may be slightly in error as to the exact value as a result of the initial reading and calibration difficulties.

Specimen Fabrication

The technique developed for the fabrication of four-inch diameter by eight-inch high specimens was very satisfactory. Initial trials using this procedure on approximately fifty specimens indicated that the variation in density throughout a specimen was generally less than 0.5 percent of the average density of the cut sections. Asphalt contents were found to vary less than 0.3 percent from the average of the asphalt contents of the sections.

Some difficulties were encountered in using the kneading compactor at low foot pressures. Feeding the material into the mold during compaction leads to a drifting of the applied air pressure from the preset amount. The result was that a continual adjustment of the air pressure was necessary. At high foot pressures a similar phenomena was noted though to a lesser extent.

It was obvious from observations during compaction and by inspection of the compacted specimens that some aggregate breakdown occurred during the compaction process. The extent of this breakdown was not investigated.

During the early stages of the testing program, it was noted that uniformity of density between the three specimens prepared at one time was not always as good as desired. As a result, four specimens were

made for the various combinations of asphalt content and compaction pressure during the latter period of testing. The three specimens having the best uniformity of density were selected for testing while the fourth was held in reserve for a re-test that might be necessitated by membrane failure, equipment leakage, etc.

Effect of Test Procedure on Observed Strength

The first routine test series were performed with specimens prepared at 4 and 5 percent asphalt content using compaction pressures of 150 and 400 psi. Tests were performed on similar specimens using three different conditions of drainage. Quick or "Q" tests were performed in which the specimen drainage valve was closed during the application of all load increments. Consolidated-Quick or "Qc" tests were accomplished in which the drainage valve was open during a one-hour period of confining pressure application and then closed for the application of all axial load increments. The Consolidated-Partially Drained or "CP" tests were completed in a fashion similar to the "Qc" tests except that the drainage valve was kept open during the application of all loads.

In no case was there sufficient difference between the results of "Q", "Qc", and "CP" tests for specimens prepared at the same asphalt content and compaction pressure to be noticeable (see Figures 16-21 and Table 7, Appendix A). Variations in deviator stress of approximately 10 percent were observed but these did not follow a pattern that could be satisfactorily correlated with drainage conditions. It was assumed, therefore, that these differences were random variations due to non-uniformity in the test specimens or to other uncontrolled test errors.

The effect of drainage was found to be equally unnoticeable for tests performed on three specimens using a slow rate of loading (see tests 63-65, Table 7, Appendix A).

The results of the first thirty-nine tests indicated little or no difference in the strength or stability of similar specimens tested under different conditions of drainage. The remainder of the tests were performed, therefore, using the "Q" test procedure only.

Evidence is plentiful to indicate that negative pore pressures do develop within the specimens during testing. An expansion of the specimen under conditions of no drainage implies an increase in the volume of air within the sample and, therefore, a negative pore pressure. It was noted also that if the drainage valve was opened at the completion of a "Q" or "Qc" test there was an obvious sound of moving air. That this air was moving into the specimen was evidenced by a suction developed on the operator's hand if held over the drainage outlet.

It must be concluded, in light of evidence of negative pore pressure and uniformity of observed strength results regardless of drainage conditions, that, though pore pressures apparently develop, their effect was so small compared to the strength of the mixture that they did not influence the test results. This conclusion appears to be the only rational one possible for loading times of up to one hour between load increments.

Volume Changes

In every specimen tested, regardless of test procedure or initial conditions, a change in volume of the specimen was noted. All tests followed the same general pattern of volume change. Typical curves of

volume change with axial strain are shown in Figures 12 and 13. The specimen decreased in volume under the application of the confining pressure with the magnitude of the decrease a function of the magnitude of the confining pressure all other things being equal. Further, but smaller, decreases in volume occurred under each of the first few axial load increments. Additional axial loading produced a volume increase that continued to failure. The magnitude of the volume changes that occurred at various points throughout a test are tabulated in Table 7 (Appendix A).

The amount of the reduction in specimen volumes was small. In nearly every case the volume decrease amounted to less than one-half of one percent of the specimen volume less the volume of surface voids. In all cases, the axial strain at the point of minimum specimen volume was less than that at the Proportional Limit. In nearly all cases, the volume change increased linearily with axial strain over the range of strain values from slightly less than those at the Proportional Limit to values at failure.

The slight reduction in specimen volume noted during the early portion of all tests was thought to be due to several causes. A portion of the reduction was possibly due to the recovery of expansion that may have occurred upon release of compaction pressures applied during specimen fabrication. Another portion of the reduction may have been due to elastic compression of specimens under the confining pressures. In addition, some decrease in volume may have occurred as a result of plastic deformation of the bitumen.

It seems apparent that dilatancy occurs with bituminous mixtures subjected to triaxial shear. This can be explained in a manner similar to that utilized for explaining the same phenomena in dense-granular soils. As shearing begins to occur, grains are forced to move apart in order to ride over one another. This expansion continues until all particles on the shearing plane are free to move. At this point the maximum strength has been reached. The dilatancy or volume increase with axial strain occurs at a uniform rate during shearing and its value is a function of the initial density of the aggregate mass.

A study of the volume change gradients developed between points corresponding to zero percent volume change and two percent axial strain (see Table 7, Appendix A) showed the following effects for the bituminous mixtures studied: The magnitude of dilatancy increased with increasing compaction pressure regardless of asphalt content. At a constant compaction pressure it was observed that the rate of volume increase with axial strain was greatest for the lower asphalt contents. The former result is as expected based on experience with dense-graded granular soils. The latter observation is not explained so readily. It appears probable that the magnitude of dilatancy is less for the higher asphalt contents because of the presence of thicker films of the viscous binder between particles. This would permit the specimen to accept greater axial strains with less movement of the aggregate particles than if the asphalt film were of less thickness as is the case for the lower asphalt contents. The asphalt would permit plastic deformation of the mixture and would reduce the movement necessary for particles to move over one another.

Relationships Between Void Content and Strength

Computations of the void ratio and percent voids at two percent strain and the corresponding deviator stress show that there is a definite relationship between voids and strength for the mixtures and conditions studied. Figures 28, 29, 30, and 31 illustrate this relationship for both void ratio and percent voids for tests performed at all conditions of drainage on specimens prepared at 4 and 5 percent asphalt content and 150 and 400 psi compaction pressure.

With limited deviation, the relationship of void ratio and percent voids, both at two percent strain and the Proportional Limit, is linear with the logarithm of the deviator stress. This linear relationship holds for all tests on specimens prepared by the same compaction pressure and at the same asphalt content regardless of confining pressures or drainage conditions during the test. The two percent strain curves indicate that the linear relationship appears to be valid for cases where the values of percent voids and void ratio are both larger and smaller than the initial values of the void ratio and percent voids. Indications are that the slope of the straight line is larger for specimens of the lower asphalt content at any given compaction pressure than for specimens of the higher asphalt content and the difference between slopes for two asphalt contents becomes less at the lower compaction pressure.

In order to verify the uniqueness of the established relationship, two samples were prepared at 4 percent asphalt content and 400 psi compaction pressure and water saturated prior to testing. The strength-voids relationships of specimens fabricated at 4 percent asphalt content

and 400 psi compaction pressure are expressed by those of Figures 30 and 31. The water saturation served to reduce the amount of air in the specimens to a quantity essentially equal to that in specimens prepared by the same compaction pressure but at 5 percent asphalt content. Thus the void ratio values of the water-saturated specimens remained nearly equal to those of similar specimens without water saturation, while the percent voids values were reduced as though the asphalt content had been increased. Referring to Figure 31, it is noted that the percent voids -- strength relationship for the water-saturated specimens fits the relationships for 5 percent asphalt content very well for both two percent strain and Proportional Limit cases. Figure 30 shows that there is reasonable correlation of void ratio and strength at two percent strain for the water-saturated specimens with that of similar but unsaturated specimens. The relationship of void ratio and strength at the Proportional Limit for the water-saturated specimens shows some deviation from the established curve.

The deviation of the void ratio data for the water-saturated specimens from the curve established in Figure 30 is probably due to the increase in void ratio resulting from the slight increase in specimen volume as a result of water saturation. This increase in volume was considered in the computations of the water-saturated specimens test data, but the effect was to change the initial void ratio conditions slightly from those of specimens not saturated.

The diagonal straight lines of Figures 32 and 33 show the same relationship between percent voids and deviator stress at two percent strain as shown in Figures 29 and 31 and also present similar data for

routine tests at other combinations of asphalt content and compaction pressure. In addition, Figures 32 and 33 show that apparently there is a unique relationship between percent voids and deviator stress at two percent strain for a given confining pressure and compaction pressure regardless of the asphalt content at which the specimens were prepared.

A study of Figure 33 suggests the possibility of a critical value, with respect to strength, of the percent voids at two percent strain. In particular, the curves for 500 psi compaction are sufficiently curved to indicate the likelihood of a peak value of deviator stress occurring between approximately four and six percent voids. As compaction pressure was decreased the curvature of the plot of percent voids versus deviator stress at a particular confining pressure decreased until a linear relationship is indicated for the 150 psi condition. Unfortunately, there are insufficient data to clearly define the concept of a critical percent voids with respect to strength.

The implication of Figures 32 and 33 is that a family of curves may be developed for a given aggregate, a given asphalt, and an established compaction and triaxial test procedure that will define the deviator stress and percent voids at two percent axial strain. As there is a mathematical relationship between percent voids and void ratio, a similar set of curves could be developed using void ratio in lieu of percent voids.

It should be noted that the above results and conclusions are applicable only to the aggregate, aggregate gradation, and penetration grade asphalt cement used in this study as well as for the particular compaction and testing procedures used. It is felt, however, that

similar results for other aggregates, aggregate gradations, and asphalt cements are probable.

Variation of Strength With Asphalt Content

In the interest of expanding the general knowledge of the relationship between asphalt content and compressive strength under conditions of triaxial compression testing, data from all routine tests are presented in a plot between these variables in Figures 24 and 25. Superimposed over the strength curves in these figures are curves expressing the relationship between aggregate and bulk densities and asphalt content.

From Figure 24 it may be noted that the variation in strength with asphalt content at any given confining pressure is quite low for the 150 and 250 psi compaction pressures. In light of the variation in bulk density and aggregate density for the same conditions, this figure provides further verification of the lack of correlation between specimen density and strength.

Figure 25 presents similar data for 400 and 500 psi compaction pressures. The variation in strength with asphalt content at any given confining pressure is noticeable in this figure. Essentially equal strength values were developed under the 400 psi compaction pressure at 3 and 4 percent asphalt contents, but there was a rather sharp decrease in strength at the 5 percent asphalt content condition. This decrease is pronounced even though bulk density and aggregate density show a continuing increase with asphalt content. The 500 psi compaction results again show about the same strength values for the two lower confining pressures; however, a strength decrease is noticeable from 3 to 4 percent asphalt content for the 60 psi confining pressure. It is apparent also

that the sharp decrease in strength at the 5 percent asphalt content is reflected by a decrease in aggregate density even though bulk density continued to increase.

On the basis of Figures 24 and 25 it is apparent that compaction pressure greatly affects the observed compressive strength of a specimen formed from a given mixture. It is apparent also that a density criterion for suitability of a mixture from a strength standpoint is not reliable. Furthermore, it appears that the aggregate density of a specimen is a more significant measure of the change of strength than is bulk density.

The apparent shear strength parameters, "c" and "Ø", were established by a Mohr's circle of stress analysis for the failure conditions and for the stress at two percent axial strain for all test series. Typical Mohr circles for each of these strength criteria are shown in Figures 22 and 23. In all cases the envelope of stress was fitted to the circles of stress as a straight line. In some cases the tangency of the circles to the fitted straight line was not perfect (see Figure 23). The variation from a straight line envelope, however, was so slight that a curved envelope did not appear to be justified. It is conceivable that further tests at higher confining pressures would emphasize the possibility of a curved envelope of failure.

Figure 26 and Table 10 (Appendix A) present the results of a Mohr's circle analysis of the shear strength parameters at the failure condition for all routine tests. It may be readily observed that there is a decided decrease in the value of the observed angle of internal friction with increased asphalt content at the two higher compaction pressures.

This decrease is as expected when it is realized that asphalt film thickness increases with asphalt content and thus tends to reduce the intimate contact between aggregate particles. At the two lower compaction pressures there is little change in the observed angle of internal friction with changes in asphalt content. In fact there appears to be a slight increase in this parameter as asphalt content increases. It is thought that the aggregate density under these compaction pressures is such that an increase in asphalt content serves to improve compaction efficiency and that a sharp decrease in observed angle of internal friction would occur at still higher asphalt contents where these low compaction pressures are used.

The cohesion parameter relationships shown in Figure 26 follow an interesting pattern. The following comments represent an attempt to explain these relationships utilizing the concepts advanced by other investigators. As an example, see Herrin (70). As compaction pressure increases the variation of cohesion with asphalt content varies gradually from essentially no change to a curve that is markedly convex. For a given asphalt content the value of cohesion increases with compaction pressure for 3 and 4 percent asphalt contents but the cohesion at 5 percent asphalt content decreases upon an increase in compaction pressure from 400 to 500 psi compaction. Apparently the increase in cohesion that occurs with an increase in compaction pressure results from the higher compaction pressure forcing the coated particles into more intimate contact and increasing the contact area between coated particles. If the density of the specimen is increased by increased compaction pressures to such a point that the asphalt tends to develop thicher films between particles, the particles are forced apart and the contact area

between the coated particles is reduced. This is apparently the case for the change of compaction pressure from 400 to 500 psi for 5 percent asphalt content. The result is a reduction in cohesion.

For a given compaction pressure an increase in asphalt content acts to increase lubrication and to facilitate compaction. This in turn increases the contact area between coated particles and in so doing increases the value of cohesion to a maximum. Beyond the critical value of asphalt content, additional bitumen tends to form thicker films on the particles and to force them apart. The result is a decrease in contact between the coated particles and in the value of cohesion. The foregoing explanation of the effect of asphalt content on cohesion at a given compaction pressure is clearly illustrated at 400 and 500 psi compaction pressures in Figure 26. The explanation, however, does not appear to apply to the relationship exhibited by the two lower compaction pressures. It is thought that further increases in asphalt content beyond the quantities used in this study at the low compaction pressures would result in a critical value of asphalt content and a decrease in cohesion for asphalt contents beyond the critical value.

It is quite likely that apparent cohesion is not the proper parameter to use for expressing the relationships between cohesion, asphalt content, and compaction pressure. Future studies may permit the establishment of the Hvorslev true cohesion parameter and this concept possibly will provide further understanding of these relationships.

Modulus of Deformation

Figure 15 and Tables 8 and 9 (Appendix A) present the results of the computation of the Modulus of Deformation for all tests performed. The term Modulus of Deformation (C_d) has been used to define the ratio of the deviator stress to axial strain at a point in keeping with the

established terminology of flexible pavement design. For this study, the point at which the Modulus of Deformation is defined is the Proportional Limit.

These data are included primarily to provide further information on the order of magnitude of the Modulus of Deformation for bituminous mixtures and to indicate, in general, the trend in the variation of this measure of strength with asphalt content and compaction pressure.

From Figure 15 it is interesting to note that at the low compaction pressure there is very little difference in the measured Modulus of Deformation for 4 and 5 percent asphalt contents. The values obtained were so similar that a single curve is fitted to the data for both asphalt contents. However, there is a marked difference in the Modulus of Deformation for the two asphalt contents at the higher compaction pressure. It would appear that the Modulus of Deformation is primarily a function of the aggregate density at the lower compaction pressure. At the higher compaction pressure both the strength and strain at the Proportional Limit were affected by the asphalt content in such a manner as to yield much lower values of the Modulus of Deformation for the higher asphalt content. It is quite likely that the results at the higher compaction pressure are due to the thickness of the asphalt film between particles — the greater the film thickness the less the Modulus of Deformation.

Deformation Ratio

Little or no information is available in the literature on the value of Poisson's Ratio for bituminous mixtures resulting from direct measurement. With the realization that data of the type gathered in

this study could provide some information on the subject, a limited investigation was initiated. The strict definition of Poisson's Ratio is not applicable to non-elastic materials; therefore, the term Deformation Ratio was defined in an analogous fashion to Poisson's Ratio for the purpose of extending the scope of the Poisson's Ratio concept.

Figure 34, Tables 11 and 12 (Appendix A) and Appendix B present the results and computations used for the investigation. The approach to the study is based on a scheme suggested by Jakobson (69) in a study of the Poisson's Ratio of sands.

It is thought that the developed Deformation Ratio data has two significant uses. Primarily, the data are intended to show the order of magnitude of the ratio of the average lateral strain to the axial strain that might be expected during the compression of a dense-graded bituminous mixture. Secondly, the data show the need for strain to develop in order to develop frictional shearing resistance.

In all cases studied it is apparent that the angle of shearing resistance is mobilized gradually with a continuing increase in the Deformation Ratio. This increase is continued to a point at which maximum frictional resistance is developed. With further deformation the frictional resistance tends to decrease or, at best, the frictional resistance remains constant.

The values of the angle of shearing resistance at which the maximum frictional shearing resistance is mobilized should represent the apparent angle of internal friction of the material at failure. In the four cases investigated, the range of the maximum value of the angle of shearing resistance for the three tests composing each case checked

very well with the apparent angle of internal friction established for each case by the Mohr circle of failure analysis. (See Figures 34 and 26 and Tables 11 and 12, Appendix A).

SUMMARY OF RESULTS

The following comments serve to recapitulate the findings of this study.

1. The controlled temperature room used to maintain a constant temperature of 25°C (77°F) was successful in accomplishing this purpose within a variation of ± 0.5°C.

2. The equipment developed for the measurement of volume changes of bituminous-mixture specimens under triaxial compression performed satisfactorily. Some difficulty was experienced in establishing calibration values for the change in volume of the apparatus and the triaxial cell under the application of the various confining pressures.

3. A technique for fabricating four-inch diameter by eight-inch high specimens of a bituminous mixture by the use of the kneading compactor was established. The scheme developed provided specimens that were uniform with respect to density generally within 0.5 percent of the average density of the sections cut from the specimen and uniform with respect to asphalt content to within 0.3 percent of the average asphalt content of the sections.

4. Tests on similar specimens indicated that the three drainage conditions used had no measured effect on the magnitude of observed strength values even though there was indirect evidence of the development of a negative pore pressure during the tests. This finding was established for a five-minute and a one-hour time interval between

the application of axial load increments.

5. It was established that volume changes occur during the testing of the particular bituminous mixtures used and that these changes consist of an initial compression of generally less than one-half of one percent of the specimen volume followed by a continual expansion to failure. This result indicates that dilatancy occurs in dense-graded bituminous mixtures in a fashion similar to that observed in dense-granular soils.

6. For the particular bituminous mixtures tested, a relationship between the void ratio and percentage voids of a specimen, at two percent strain and at the Proportional Limit, and the deviator stress at these points was established. This relationship was found to be linear for a semi-log plot, dependent upon the initial voids characteristics of the specimen, and apparently independent of the test confining pressure.

7. The unique character of the voids-deviator stress relationship was verified by tests of specimens compacted to given initial conditions and then having these conditions changed by water saturation prior to testing.

8. The effects of compaction pressure and asphalt content on the apparent shear strength parameters "c" and "Ø", and on the compressive strength, bulk density and aggregate density were investigated for a particular aggregate blend, compaction procedure, and method of test. The findings of this portion of the study show that both the apparent angle of internal friction and cohesion are affected not only by asphalt content but by compaction pressures as well. Indications are

that aggregate density variation provides a fairly good measure of relative values of observed compressive strengths while bulk density does not.

9. Values of the Modulus of Deformation established at the Proportional Limit for the bituminous mixtures tested are presented. The data developed indicate that the Modulus of Deformation was affected only slightly by asphalt content at a low compaction pressure. The effect of asphalt content on specimens prepared at the high compaction pressure was pronounced. The data show that there is a marked effect of compaction pressure on the Modulus of Deformation.

10. The results of a limited investigation of the Deformation Ratio of the bituminous mixtures used indicate that the angle of shearing resistance of a mixture is mobilized gradually with a continuing increase of Deformation Ratio to a point at which the maximum shearing resistance is developed. Further deformation tends to decrease the developed frictional resistance. The maximum angle of shearing resistance defined by the Deformation Ratio studies compares favorably with the apparent angle of shearing resistance established by the Mohr circle of failure analysis.

CONCLUSIONS

The results obtained from the analysis of test data from sixty-five triaxial tests performed on specimens prepared at four different compaction pressures and three different asphalt contents appear to justify the following conclusions. It should be realized that these conclusions are applicable only to the particular bituminous mixtures and compaction procedures, and for triaxial compression tests of the types used in this study. Furthermore, it should be noted that all percent voids relationships are based on a consideration of the asphalt available to fill the mineral aggregate voids rather than upon the total asphalt used in mixing.

Differences in the drainage conditions existing during the performance of triaxial compression tests on dense-graded bituminous mixtures do not materially affect strength values. This conclusion appears justified for time intervals between load applications of one hour or less.

Indirect evidence indicates that a negative pore pressure develops in the gaseous phase of a dense-graded bituminous mixture subjected to triaxial compression. The indicated negative pore pressure did not affect the observed strength results, so it must be concluded that the effect of this pressure is so small as to be masked by the high strength values of the mixtures tested.

All test results clearly demonstrate that the bituminous mixture specimens tested changed volume during the progress of a test and that the volume change consists of a slight reduction in volume followed by an expansion to failure.

Within the range of the materials and procedures used in this study, there appears to be a unique relationship between the percentage voids and/or the void ratio and the deviator stress at two percent axial strain and at the Proportional Limit. The established relationship apparently depends upon the initial conditions of void ratio and percentage voids to which the specimen is compacted. The results of tests on specimens with the initial conditions varied from the as-compacted values by water saturation appear to justify the established relationship.

Indications of the effect of compaction pressure and asphalt content on the observed apparent shear strength parameters and on the compressive strength of the test mixtures were established. It was found that variation in measured cohesion values depend upon compaction pressure as well as on asphalt content and that the variation with asphalt content becomes more pronounced at the higher compaction pressures. Similar effects are established for the apparent angle of shearing resistance and for compressive strength at failure. It is clearly shown that variations in bulk density do not necessarily reflect similar variations in strength but there are indications that variations in aggregate density are reflected by similar variations in strength.

The Modulus of Deformation defined at the Proportional Limit is affected by asphalt content at high compaction pressures but apparently is independent of asphalt content at low compaction pressures.

A limited number of computations of the Deformation Ratio of the mixtures tested indicates that this measure varies from values below 0.5 to nearly 1.0 during the progress of a test. Further, it is shown that Deformation Ratio values of greater than 0.5 are necessary in order for full frictional shearing resistance to be mobilized.

SUGGESTIONS FOR FURTHER RESEARCH

The triaxial test offers the most versatile tool for the investigation of the fundamental strength characteristics of bituminous mixtures. Numerous variables exist and, at the present time, there is no accepted method of separating these variables and evaluating their effects on strength characteristics. The study herein reported has attempted to open new fields of thought in the analysis of bituminous mixtures. There are numerous investigations that need to be undertaken to further the basic knowledge of such mixtures. The following suggestions constitute only a partial listing of needed research projects.

1. It is apparent to anyone using the kneading compactor that aggregate degradation occurs during the compaction process. The extent of this breakdown, the sizes of particles affected, the part played in the process by the asphalt content, and the relationship of the degradation occurring in laboratory compaction to that during job mixing, placement, and service use should be investigated.

2. Additional testing to verify the unique relationships suggested by this study between void ratio and/or percent voids and deviator stress at a specified stability criterion is needed. In particular, different aggregate blends should be studied to verify whether or not the suggested relationships will hold for mixtures other than the ones used in this investigation. A study similar to the one

reported but utilizing open-graded mixtures would be worthwhile in furthering the knowledge of mixtures of this type.

3. It would be desirable to investigate further the effects of test drainage conditions on strength for increment loading periods in excess of one hour. The apparatus used for the reported study would not be suitable for such long-time test investigations.

4. It would be of particular interest to study the volume change-strength relationships for a bituminous mixture that has yielded a curved envelope of failure under conditions of testing and analysis other than those used in this study such as constant rate of deformation tests. Such an investigation would cast additional light on the explanation of the curved envelope of failure phenomena.

5. Investigations of the relationship of the as-compacted or initial voids characteristics to the value of these characteristics at a suitable strength criterion would be a valuable contribution to the progress of the development of a rational mixture design procedure.

6. The data obtained in this study, as well as that from many others, shows very clearly the need for knowledge of the magnitude of confining pressure that best corresponds to the confinement existing on a section of bituminous mixture subject to full-scale loading. The widespread use of a triaxial test method of mixture design will probably be delayed until such information is available.

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APPENDIX A

TABLE 7

Summary of Results

Tests 1 - 24

Test Number	Type of Test	Confining Pressure, psi	Asphalt Content, %	Compaction Pressure, psi	Initial Conditions			
					Height cm.	Diameter cm.	Density pcf.	Aggregate Density, pcf.
1	Q	15	2.964	150	20.93	10.18	143.27	139.03
2	Q	30	2.964	150	20.90	10.18	143.33	139.09
3	Q	60	2.963	150	20.92	10.18	143.39	139.15
4	Q	15	3.996	150	20.83	10.18	145.27	139.46
5	Q	30	3.999	150	20.70	10.18	146.56	140.52
6	Q	60	3.996	150	20.70	10.18	146.50	140.46
7	Qc	15	3.993	150	20.75	10.18	146.00	139.90
8	Qc	30	3.994	150	20.88	10.18	145.56	139.53
9	Qc	60	3.997	150	20.70	10.18	145.56	140.46
10	CP	15	3.997	150	20.83	10.18	145.50	139.46
11	CP	30	3.997	150	20.73	10.18	146.06	140.03
12	CP	60	3.997	150	20.78	10.18	145.75	139.71
13	Q	15	5.004	150	20.73	10.18	147.14	139.78
14	Q	30	5.001	150	20.64	10.18	148.25	140.59
15	Q	60	5.004	150	20.60	10.18	148.44	140.77
16	Qc	15	5.005	150	20.68	10.18	147.76	140.34
17	Qc	30	5.006	150	20.67	10.18	147.70	140.34
18	Qc	60	5.006	150	20.60	10.18	148.14	140.71
19	CP	15	5.006	150	20.72	10.18	147.26	139.90
20	CP	30	5.006	150	20.72	10.18	147.33	139.90
21	CP	60	5.006	150	20.60	10.18	148.01	140.59
22	Q	15	2.956	250	20.57	10.18	146.14	141.83
23	Q	30	2.961	250	20.53	10.18	146.14	141.83
24	Q	60	2.958	250	20.60	10.17	146.20	141.83

TABLE 7 (cont'd)

Summary of Results

Tests 25 - 48

Test Number	Type of Test	Confining Pressure, psi	Asphalt Content, %	Compaction Pressure, psi	Initial Conditions			
					Height cm.	Diameter cm.	Density pcf.	Aggregate Density, pcf.
25	Q	15	3.998	250	20.41	10.18	148.01	142.08
26	Q	30	3.997	250	20.41	10.18	148.20	142.27
27	Q	60	3.994	250	20.41	10.18	148.26	142.33
28	Q	15	5.009	250	20.19	10.18	151.01	143.39
29	Q	30	5.009	250	20.13	10.18	151.44	143.83
30	Q	60	5.007	250	20.17	10.18	151.07	143.52
31	Q	15	2.968	400	20.32	10.18	147.39	143.02
32	Q	30	2.968	400	20.25	10.18	147.95	143.58
33	Q	60	2.967	400	20.24	10.18	148.07	143.64
34	Q	15	3.997	400	20.22	10.18	149.70	143.71
35	Q	30	4.020	400	20.17	10.18	150.07	144.02
36	Q	60	3.996	400	20.08	10.18	150.51	144.52
37	Qc	15	3.995	400	20.10	10.18	150.63	144.64
38	Qc	30	3.993	400	20.10	10.18	150.45	144.46
39	Qc	60	3.996	400	20.03	10.18	150.95	144.95
40	CP	15	3.996	400	20.10	10.18	150.51	144.52
41	CP	30	3.996	400	20.15	10.18	150.13	144.14
42	CP	60	3.997	400	20.12	10.18	150.38	144.39
43	Q	15	5.005	400	19.90	10.19	152.94	145.27
44	Q	30	5.006	400	19.90	10.19	152.82	145.20
45	Q	60	5.006	400	19.88	10.19	152.82	145.14
46	Qc	15	5.003	400	19.94	10.19	152.69	145.02
47	Qc	30	5.005	400	19.88	10.19	152.94	145.27
48	Qc	60	5.006	400	19.87	10.19	153.00	145.33

TABLE 7 (cont'd)

Summary of Results

Tests 49 - 65

Test Number	Type of Test	Confining Pressure, psi	Asphalt Content, %	Compaction Pressure, psi	Initial Conditions			
					Height cm.	Diameter cm.	Density pcf.	Aggregate Density, pcf.
49	CP	15	5.005	400	19.92	10.19	152.82	145.14
50	CP	30	5.006	400	19.88	10.19	152.88	145.27
51	CP	60	5.006	400	19.92	10.19	152.94	145.27
52	Q	15	2.968	500	20.00	10.18	149.82	145.39
53	Q	30	2.967	500	19.97	10.18	150.01	145.58
54	Q	60	2.967	500	19.96	10.18	150.13	145.64
55	Q	15	3.999	500	19.83	10.18	152.32	146.20
56	Q	30	3.999	500	19.82	10.18	152.51	146.39
57	Q	60	3.999	500	19.83	10.18	152.51	146.45
58	Q	15	5.007	500	19.80	10.18	153.69	146.02
59	Q	30	5.010	500	19.77	10.19	153.69	146.02
60	Q	60	5.010	500	19.77	10.19	153.69	145.95
61	Q*	15	3.996	400	20.15	10.25	150.63	142.08
62	Q*	60	3.999	400	20.18	10.24	150.76	142.15
63	Q**	30	3.995	400	20.02	10.18	151.07	145.02
64	Qc**	30	3.996	400	20.01	10.18	151.26	145.27
65	CP**	30	3.996	400	20.09	10.18	150.57	144.58

* Water saturated prior to testing

** Slow rate of loading

TABLE 7 (cont'd)

Summary of Results

Tests 1 - 24

Test Number	Compressive Strength, psi				Axial Strain, %			
	P.L.	2% str.	failure	0% vol. str.	P.L.	failure	min. vol.	0% vol. str.
1	120	172	182.5	150	0.64	3.77	0.24	0.64
2	145	223	231	239	0.53	3.66	0.31	0.80
3	192.5	320	341	401	0.55	4.00	0.45	1.32
4	112.5	180	190	125	0.58	3.33	0.26	0.68
5	147.5	238	252.5	190	0.60	3.90	0.30	0.92
6	192.5	322	357.5	292.5	0.50	4.08	0.47	1.41
7	115	185	195	140	0.60	4.83	0.25	0.83
8	125	242.5	253	193	0.45	3.92	0.37	0.97
9	200	332.5	360	293	0.56	4.15	0.30	1.24
10	110	180	190	147	0.63	3.45	0.22	1.00
11	145	237.5	247	207	0.60	3.80	0.45	1.17
12	200	322	342.5	310	0.60	3.90	0.40	1.66
13	100	179	186	125	0.54	4.15	0.20	0.74
14	132.5	222.5	233	177.5	0.56	3.44	0.32	0.98
15	197.5	313	340	300	0.65	4.40	0.47	1.67
16	107.5	195	201	158	0.51	2.94	0.20	0.98
17	130.5	237	252	192.5	0.52	3.84	0.35	1.02
18	202.5	331	358	315	0.60	4.69	0.40	1.65
19	115	182.5	188	135	0.60	3.45	0.27	0.81
20	135	227	249	197	0.60	4.45	0.30	1.15
21	210	345	378	351	0.64	4.10	0.45	2.15
22	102.5	202	203	132.5	0.35	2.40	0.20	0.48
23	145	262	266	198	0.42	2.90	0.30	0.68
24	202.5	362	372	313	0.44	3.15	0.41	1.00

TABLE 7 (cont'd)

Summary of Results

Tests 25 - 48

Test Number	Compressive Strength, psi				Axial Strain, %			
	P.L.	2% str.	failure	0% vol. str.	P.L.	failure	min. vol.	0% vol. str.
25	120	206	208	138	0.45	4.50	0.21	0.56
26	150	262	267.5	209	0.49	4.40	0.32	0.86
27	220	373	385	331	0.56	4.30	0.46	1.20
28	115	203	207	142	0.45	3.15	0.25	0.64
29	150	257.5	261	198.5	0.50	2.60	0.30	0.73
30	207.5	365	382	327.5	0.54	3.90	0.44	1.31
31	165	245	247.5	121	0.57	2.20	0.26	0.51
32	195	323	327	177.5	0.58	2.66	0.25	0.67
33	237	441	447	289	0.50	2.64	0.40	1.31
34	165	259	262.5	162.5	0.50	2.90	0.20	0.49
35	205	324	334	237.5	0.52	3.65	0.25	0.66
36	285	442.5	452	395	0.60	3.16	0.40	1.17
37	160	250	251	172.5	0.53	3.44	0.19	0.60
38	202.5	322	325	250	0.52	3.39	0.25	0.76
39	290	440	450	371	0.60	3.12	0.40	1.04
40	170	250	251	185	0.51	2.65	0.17	0.59
41	230	326	331	251	0.62	3.00	0.25	0.72
42	290	439	450	379	0.60	3.40	0.34	1.05
43	98	182	224	114	0.73	4.33	0.31	0.93
44	157.5	250	272	180	0.88	4.15	0.38	1.05
45	205	340	421	331	0.92	4.28	0.53	1.90
46	97.5	190	225	140	0.69	4.45	0.35	1.19
47	155	237.5	278	203	1.05	5.10	0.55	1.52
48	205	323	405	318	1.03	4.73	0.63	1.95

TABLE 7 (cont'd)

Summary of Results

Tests 49 - 65

Test Number	Compressive Strength, psi				Axial Strain, %			
	P.L.	2% str.	failure	0% vol. str.	P.L.	failure	min. vol.	0% vol. str.
49	97.5	198	220	137.5	0.70	4.19	0.34	1.06
50	165	253	276	205	0.93	4.20	0.40	1.25
51	207.5	302.5	392	326	1.17	5.00	0.60	2.30
52	210	298	298	132.5	0.50	2.00	0.15	0.43
53	255	382.5	385	276	0.56	2.60	0.25	0.63
54	360	548	568	452	0.67	2.85	0.37	1.00
55	167	302	305	205	0.41	1.65	0.20	0.56
56	215	371	374	270	0.55	2.44	0.34	0.74
57	315	519	525	421	0.64	2.70	0.48	1.01
58	115	143	196	108	1.49	5.10	0.38	1.40
59	140	172.5	252.5	156	1.56	5.85	0.60	1.77
60	213	215	360	258	1.96	5.85	0.87	2.56
61	130	205	220	180	0.90	3.75	0.50	1.47
62	210	370	404	385	0.76	3.50	0.70	2.30
63	167.5	279	282.5	248	0.45	3.40	0.30	0.88
64	170	275	278	241	0.50	3.15	0.29	1.00
65	160	274	277	246	0.44	3.00	0.35	0.95

TABLE 7 (cont'd)

Summary of Results

Tests 1 - 24

Test Number	Volume Change - cc, (%)			Percent Voids			Void Ratio		
	P.L.	2% str.	min. vol.	initial	P.L.	2% str.	initial	P.L.	2% str.
1	-1.0(+0.01)	+22.0(+1.28)	-2.8(-0.15)	9.97	9.82	11.12	0.1766	0.1745	0.1915
2	-2.0(-0.12)	+14.4(+0.85)	-3.0(-0.18)	9.94	9.60	10.40	0.1761	0.1717	0.1827
3	-4.2(-0.24)	+ 5.6(+0.33)	-4.4(-0.25)	9.91	9.40	9.65	0.1758	0.1675	0.1726
4	-0.7(-0.05)	+20.0(+1.40)	-2.3(-0.14)	7.31	7.08	8.25	0.1731	0.1705	0.1834
5	-2.6(+0.14)	+12.2(+0.81)	-3.3(-0.20)	6.62	6.26	7.16	0.1645	0.1620	0.1708
6	-4.5(-0.28)	+ 5.0(+0.28)	-4.5(-0.28)	6.67	5.79	6.30	0.1650	0.1541	0.1640
7	-2.5(-0.14)	+16.3(+1.03)	-4.5(-0.27)	7.01	6.72	7.62	0.1693	0.1659	0.1760
8	-3.9(-0.26)	+11.0(+0.74)	-4.5(-0.26)	7.27	6.75	7.65	0.1725	0.1664	0.1800
9	-4.2(-0.30)	+ 6.0(+0.41)	-4.9(-0.30)	6.62	5.80	6.40	0.1645	0.1545	0.1630
10	-4.5(-0.31)	+14.8(+0.89)	-6.5(-0.42)	7.29	6.75	7.85	0.1730	0.1662	0.1792
11	-3.5(-0.24)	+12.4(+0.70)	-4.2(-0.25)	6.94	6.47	7.23	0.1685	0.1628	0.1724
12	-6.9(-0.42)	+ 3.2(+0.20)	-7.3(-0.44)	7.15	5.95	6.49	0.1712	0.1559	0.1632
13	-2.2(-0.12)	+17.3(+1.01)	-6.1(-0.22)	4.69	4.05	5.25	0.1704	0.1638	0.1766
14	-3.0(-0.18)	+10.6(+0.67)	-3.9(-0.23)	4.14	3.57	4.45	0.1635	0.1566	0.1669
15	-6.5(-0.38)	+ 2.0(+0.18)	-6.7(-0.41)	4.02	3.01	3.52	0.1621	0.1501	0.1562
16	-5.0(-0.29)	+14.8(+0.94)	-6.5(-0.35)	4.32	3.80	4.90	0.1658	0.1596	0.1740
17	-4.0(-0.24)	+12.5(+0.75)	-4.5(-0.26)	4.32	3.67	4.57	0.1658	0.1581	0.1712
18	-6.5(-0.38)	+ 3.0(+0.20)	-6.8(-0.41)	4.04	3.00	3.58	0.1625	0.1501	0.1569
19	-2.1(-0.13)	+16.0(+1.08)	-3.9(-0.23)	4.63	4.24	5.15	0.1697	0.1642	0.1790
20	-4.0(-0.26)	+10.0(+0.59)	-5.5(-0.32)	4.60	3.97	4.64	0.1692	0.1616	0.1740
21	-9.4(-0.56)	- 0.05(-0.07)	-9.7(-0.58)	4.15	2.97	3.47	0.1638	0.1492	0.1561
22	-1.5(-0.09)	+28.0(+1.55)	-2.2(-0.14)	8.20	7.94	9.52	0.1536	0.1504	0.1702
23	-2.5(-0.15)	+19.1(+1.14)	-2.6(-0.16)	8.19	7.85	9.10	0.1537	0.1495	0.1642
24	-3.5(-0.21)	+10.1(+0.60)	-3.5(-0.21)	8.17	7.84	8.33	0.1534	0.1493	0.1586

TABLE 7 (cont'd)

Summary of Results

Tests 25 - 48

Test	Volume Change - cc, (%)			Percent Voids			Void Ratio		
Number	P.L.	2% str.	min. vol.	initial	P.L.	2% str.	initial	P.L.	2% str.
25	-0.9(-0.06)	+24.8(+1.52)	-2.1(-0.13)	5.54	5.34	6.80	0.1512	0.1490	0.1665
26	-3.1(-0.18)	+16.6(+1.01)	-3.5(-0.21)	5.45	5.08	6.20	0.1500	0.1458	0.1660
27	-4.1(-0.25)	+ 8.5(+0.51)	-4.3(-0.26)	5.40	4.94	5.66	0.1494	0.1438	0.1525
28	-1.0(-0.07)	+20.0(+1.26)	-1.9(-0.12)	2.22	1.86	3.23	0.1408	0.1368	0.1512
29	-1.8(-0.10)	+15.1(+0.92)	-2.5(-0.15)	1.93	1.76	2.75	0.1374	0.1356	0.1478
30	-5.0(-0.30)	+ 5.9(+0.36)	-5.2(-0.31)	2.14	1.61	2.26	0.1399	0.1339	0.1415
31	+0.6(+0.04)	+27.5(+1.69)	-1.9(-0.12)	7.38	7.35	9.52	0.1438	0.1432	0.1540
32	-1.0(-0.07)	+23.6(+1.41)	-2.6(-0.16)	7.04	6.82	9.30	0.1396	0.1369	0.1493
33	-6.4(-0.39)	+ 9.0(+0.55)	-6.5(-0.40)	6.99	6.45	7.30	0.1389	0.1324	0.1439
34	0.0(0.00)	+29.5(+1.82)	-2.1(-0.12)	4.50	4.49	6.10	0.1386	0.1388	0.1586
35	-1.4(-0.08)	+21.7(+1.32)	-2.8(-0.16)	4.22	3.85	5.80	0.1359	0.1316	0.1540
36	-4.8(-0.28)	+10.8(+0.64)	-5.3(-0.34)	3.95	3.42	4.30	0.1321	0.1257	0.1362
37	-0.3(-0.05)	+28.2(+1.73)	-3.1(-0.18)	3.87	3.70	5.10	0.1311	0.1290	0.1442
38	-2.5(-0.15)	+21.7(+1.33)	-4.0(-0.23)	3.99	3.59	5.00	0.1325	0.1277	0.1436
39	-4.4(-0.26)	+12.3(+0.76)	-5.3(-0.32)	3.67	3.34	4.30	0.1288	0.1249	0.1358
40	-0.5(-0.05)	+28.7(+1.74)	-3.1(-0.19)	3.97	3.90	5.10	0.1323	0.1314	0.1450
41	-1.5(-0.07)	+22.1(+1.33)	-3.7(-0.22)	4.22	3.95	5.30	0.1352	0.1321	0.1476
42	-4.5(-0.26)	+12.1(+0.61)	-5.4(-0.32)	4.04	3.58	4.52	0.1331	0.1276	0.1386
43	-1.2(-0.07)	+10.3(+0.57)	-2.3(-0.27)	0.96	0.73	1.44	0.1263	0.1238	0.1315
44	-1.0(-0.07)	+ 9.6(+0.55)	-2.3(-0.27)	1.01	0.75	1.40	0.1269	0.1239	0.1311
45	-5.1(-0.24)	+ 0.6(+0.05)	-5.5(-0.34)	1.02	0.50	0.95	0.1270	0.1222	0.1262
46	-2.6(-0.18)	+10.2(+0.62)	-3.6(-0.23)	1.11	0.90	1.66	0.1280	0.1259	0.1346
47	-3.4(-0.19)	+ 4.6(+0.28)	-4.7(-0.28)	0.96	0.50	1.07	0.1263	0.1220	0.1274
48	-4.9(-0.29)	+ 0.4(+0.03)	-5.1(-0.32)	0.90	0.54	0.85	0.1256	0.1216	0.1249

TABLE 7 (cont'd)

Summary of Results

Tests 49 - 65

Test Number	Volume Change - cc, (%)			Percent Voids			Void Ratio		
	P.L.	2% str.	min. vol.	initial	P.L.	2% str.	initial	P.L.	2% str.
49	-2.5(-0.16)	+11.0(+0.61)	-3.3(-0.20)	1.04	0.88	1.67	0.1271	0.1251	0.1344
50	-2.2(-0.14)	+ 8.0(+0.49)	-3.8(-0.24)	0.97	0.78	1.40	0.1264	0.1242	0.1314
51	-5.9(-0.35)	- 1.8(-0.06)	-6.6(-0.40)	0.96	0.45	0.67	0.1263	0.1204	0.1230
52	+0.7(+0.05)	+40.6(+2.50)	-1.8(-0.10)	5.87	5.87	8.10	0.1254	0.1254	0.1528
53	-0.5(-0.04)	+29.5(+1.78)	-2.4(-0.15)	5.74	5.66	7.36	0.1238	0.1228	0.1435
54	-3.2(-0.20)	+16.9(+1.02)	-4.1(-0.25)	5.69	5.44	6.59	0.1233	0.1203	0.1341
55	-1.1(-0.08)	+33.5(+2.05)	-1.7(-0.12)	2.81	2.71	4.76	0.1191	0.1178	0.1418
56	-1.4(-0.10)	+23.7(+1.50)	-2.0(-0.15)	2.68	2.51	4.03	0.1174	0.1158	0.1340
57	-2.9(-0.19)	+14.6(+0.93)	-3.3(-0.20)	2.68	2.40	3.48	0.1174	0.1144	0.1264
58	+0.6(+0.03)	+ 3.8(+0.24)	-2.1(-0.13)	0.45	0.42	0.63	0.1206	0.1203	0.1226
59	-1.1(-0.06)	+ 0.9(+0.08)	-2.8(-0.16)	0.45	0.36	0.48	0.1207	0.1197	0.1210
60	-2.5(-0.15)	- 2.4(-0.14)	-4.4(-0.26)	0.47	0.27	0.28	0.1208	0.1186	0.1186
61	-6.0(-0.40)	+ 8.0(+2.26)	-7.7(-0.47)	1.28	0.84	1.70	0.1517	0.1462	0.1564
62	-11.8(-0.71)	- 3.3(-0.20)	-12.0(-0.72)	1.20	0.37	0.90	0.1512	0.1415	0.1474
63	-4.5(-0.30)	+19.0(+1.15)	-4.8(-0.30)	3.61	3.25	4.65	0.1280	0.1240	0.1403
64	-5.4(-0.35)	+16.4(+0.99)	-6.0(-0.36)	3.47	3.07	3.98	0.1317	0.1219	0.1368
65	-5.3(-0.32)	+19.0(+1.15)	-5.5(-0.34)	3.92	3.49	4.91	0.1265	0.1266	0.1435

TABLE 8

Modulus of Deformation Data

(as plotted in Figure 15)

Type of Test	Confining Pressure psi	Asphalt Content %	Compaction Pressure psi	Proportional Limit, psi	Strain at P.L., %	C_d, psi
Q	15	4	150	112.5	0.58	19400
Q	30	4	150	147.5	0.60	25580
Q	60	4	150	192.5	0.50	38500
Qc	15	4	150	115	0.60	19170
Qc	30	4	150	125	0.45	27780
Qc	60	4	150	200	0.56	35710
CP	15	4	150	110	0.63	17460
CP	30	4	150	145	0.60	24170
CP	60	4	150	200	0.60	33330
Q	15	5	150	100	0.54	18520
Q	30	5	150	132.5	0.56	23660
Q	60	5	150	197.5	0.65	30380
Qc	15	5	150	107.5	0.51	21080
Qc	30	5	150	130.5	0.52	25100
Qc	60	5	150	202.5	0.60	33750
CP	15	5	150	115	0.60	19170
CP	30	5	150	135	0.60	22500
CP	60	5	150	210	0.64	32810
Q	15	4	400	165	0.50	33000
Q	30	4	400	205	0.52	39420
Q	60	4	400	285	0.60	47500
Qc	15	4	400	160	0.53	30190
Qc	30	4	400	202.5	0.52	38940
Qc	60	4	400	290	0.60	48330
CP	15	4	400	170	0.51	33330
CP	30	4	400	230	0.62	37100
CP	60	4	400	290	0.60	48330
Q	15	5	400	98	0.73	13420
Q	30	5	400	157.5	0.88	17900
Q	60	5	400	205	0.92	22280
Qc	15	5	400	97.5	0.69	14130
Qc	30	5	400	155	1.05	14760
Qc	60	5	400	205	1.03	19900
CP	15	5	400	97.5	0.70	13930
CP	30	5	400	165	0.93	17740
CP	60	5	400	207.5	1.17	17740

TABLE 9

Modulus of Deformation Data

Type of Test	Confining Pressure psi	Asphalt Content %	Compaction Pressure psi	Proportional Limit, psi	Strain at P.L., %	C_d, psi
Q	15	3	150	120	0.64	18750
Q	30	3	150	145	0.53	27360
Q	60	3	150	192.5	0.55	35000
Q	15	3	250	102.5	0.35	29290
Q	30	3	250	145	0.42	34520
Q	60	3	250	202.5	0.44	46020
Q	15	4	250	120	0.45	26670
Q	30	4	250	150	0.49	30610
Q	60	4	250	220	0.56	39280
Q	15	5	250	115	0.45	25550
Q	30	5	250	150	0.50	30000
Q	60	5	250	207.5	0.54	38420
Q	15	3	400	165	0.57	28950
Q	30	3	400	195	0.58	33620
Q	60	3	400	237	0.50	47400
Q	15	3	500	210	0.50	42000
Q	30	3	500	255	0.56	45530
Q	60	3	500	360	0.67	53730
Q	15	4	500	167	0.41	40730
Q	30	4	500	215	0.55	39090
Q	60	4	500	315	0.64	49220
Q	15	5	500	115	1.49	7720
Q	30	5	500	140	1.56	8970
Q	60	5	500	213	1.96	10870
Q*	15	4	400	130	0.90	14440
Q*	60	4	400	210	0.76	27630
Q**	30	4	400	167.5	0.45	37200
Qc**	30	4	400	170	0.50	34000
CP**	30	4	400	160	0.44	36390

* Water saturated prior to testing
** Slow rate of loading

TABLE 10

Shear Strength Parameters at Failure

Asphalt Content, %	Compaction Pressure, psi	Cohesion "c", psi	Angle of Internal Friction, "Ø" degrees
3	150	32	39.3
3	250	34	41.0
3	400	40	43.0
3	500	40	48.5
4	150	32*	40.3*
4	250	34	41.5
4	400	43*	42.4*
4	500	48	45.2
5	150	32*	40.8*
5	250	34	41.2
5	400	37.5*	41.8*
5	500	33	40

* Average values from the results of "Q", "Qc", and "CP" tests at the indicated asphalt contents and compaction pressures

TABLE 11

Deformation Ratio Results

	4% Asphalt Content			
	Compaction Pressure 150 psi		Compaction Pressure* 400 psi	
Confining Pressure psi	D_r	Sine ϕ_1	D_r	Sine ϕ_1
15	0.63	0.121	0.75	0.584
	0.72	0.424	0.94	0.682
	0.81	0.563	1.03	0.654
	0.90	0.621		
	0.94	0.635		
30	0.48	0.077	0.57	0.428
	0.56	0.414	0.64	0.574
	0.64	0.565	0.75	0.658
	0.71	0.612	0.92	0.705
	0.78	0.641	0.98	0.688
	0.83	0.642		
60	0.37	0.048	0.40	0.277
	0.42	0.307	0.43	0.409
	0.47	0.454	0.47	0.500
	0.51	0.532	0.52	0.566
	0.58	0.588	0.57	0.617
	0.62	0.609	0.64	0.654
	0.65	0.627	0.72	0.676
	0.69	0.640	0.81	0.681
Range of Maximum ϕ_1, degrees		39.4-40.2		43.0-44.8

* Data plotted in Figure 34

TABLE 12

Deformation Ratio Results

Confining Pressure psi	5% Asphalt Content Compaction Pressure 150 psi		Compaction Pressure 400 psi	
	D_r	Sine ϕ_1	D_r	Sine ϕ_1
15	0.63	0.249	0.64	0.335
	0.77	0.598	0.69	0.514
	0.91	0.628	0.75	0.612
			0.82	0.634
			0.87	0.644
30	0.50	0.152	0.55	0.296
	0.61	0.486	0.62	0.571
	0.71	0.601	0.65	0.617
	0.79	0.620	0.71	0.652
	0.80	0.638	0.77	0.677
			0.83	0.677
60	0.36	0.089	0.45	0.232
	0.38	0.333	0.47	0.407
	0.44	0.468	0.50	0.477
	0.51	0.541	0.53	0.563
	0.58	0.595	0.55	0.589
	0.63	0.615	0.57	0.612
	0.67	0.629	0.59	0.632
	0.71	0.630	0.62	0.650
			0.64	0.665
			0.68	0.677
			0.72	0.679
Range of Maximum ϕ_1, degrees		38.7-39.5		40.2-42.9

APPENDIX B

TABLE 13

Typical Mix Record

Specimen Q-12

	Batch 1	Batch 2
Tare Weight of Bowl, g	3141	3140
Weight of Bowl + Aggregate, g	5039	5038
Weight of Aggregate, g	1898	1898
Weight of Bowl + Aggregate + Asphalt, g	5139	5138
Weight of Asphalt, g	100	100
Weight of Mixture, g	1998	1998
Asphalt Content of Batch, %		
by weight of mixture	5.005	5.005
by weight of aggregate	5.269	5.269

Asphalt Content of Specimen, %	
by weight of mixture	5.005
by weight of aggregate	5.269

Specimen Diameter, cm	10.19, 10.19, 10.19	Average = 10.19
Specimen Height, cm	19.95, 19.90, 19.85	Average = 19.90
Specimen Volume, cc	1622.85	
Specimen Weight, g	3997.0	
Specimen Density, g/cc	2.451	
Specimen Density, pcf	152.942	

Date: 3/21/60
By: JS & TC

TABLE 14

Typical Specimen Initial Characteristics Computations

Specimen Q-12

Asphalt Content, % by weight of mix	5.005
Volume of Specimen, cc	1622.85
Weight of Specimen, g	3977.0
Weight of Aggregate, g	3777.951
Weight of Asphalt, g	199.049
% Bitumen Absorption	0.72
Weight of Asphalt Absorbed, g	27.201
Net Weight of Asphalt, g	171.848
Specific Gravity, Aggregate Blend	2.622
Specific Gravity, Asphalt	1.033
Volume of Aggregate (V_m), cc	1440.87
Volume of Asphalt (V_b), cc	166.36
Volume of Aggregate + Asphalt, cc	1607.23
Volume of Air (V_a), cc	15.62
Volume of Voids in Aggregate, cc	181.98
% V	0.96
$\%V_{ma}$	11.21
% V filled	91.42
$e = \dfrac{V_a + V_b}{V_m}$	0.1263
Length of Specimen, cm	19.90
Cross-Sectional Area, sq. cm	81.55
Aggregate Density, g/cc	2.328
Aggregate Density, pcf	145.267

Date: 3/22/60
By: JS

TABLE 15

Typical Test Data

Type of Test: "Q"
Specimen: Q-12, 5%, 400 psi

Date: 3/23/60
By: JS

Loading psi *	Elapsed Time, min.	Vol. Scale cm	δ Vol. Scale, cm	δ Vol. cc	Corr. δ V, cc	Δ V cc	Dial cm	Δ Dial cm	Piston cc
$\sigma_3 = 15$	0	37.50					0.043		
	0.25	33.40	-4.10	-20.377	-2.465	-2.465	0.035	0.008	-0.023
	5.0	33.05	-0.35	- 1.740	-1.740	-4.205	0.035		
$\Delta\sigma = 50$	10.0	33.00	-0.05	- 0.248	-0.583	-4.788	0.153	0.118	+0.335
100	15.0	33.30	+0.30	+ 1.491	+1.272	-3.516	0.236	0.077	-0.219
150	20.0	34.00	+0.70	+ 3.479	+3.203	-0.313	0.333	0.097	-0.276
180	25.0	34.95	+0.95	+ 4.722	+4.469	+4.156	0.422	0.089	-0.253
200	30.0	36.20	+1.25	+ 6.212	+5.945	+10.101	0.516	0.094	-0.267
220	35.0	38.35	+2.15	+10.686	+10.302	+20.403	0.651	0.135	-0.384
230	40.0	41.80	+3.45	+17.146	+16.580	+36.983	0.850	0.199	-0.566
240	43.0	47.00	+5.20	+25.844	+24.991	+61.974	1.150	0.300	-0.853

* Nominal stress on a 4-inch diameter specimen

TABLE 16

Typical Stress and Strain Computations

Type of Test: "Q"
Specimen: Q-12, 5%, 400 psi

Date: 3/23/60
By: JS

Total Load lbs	Friction lbs	Net Load lbs	ΔVol. cc	$V_1 + \Delta V$ cc	Δl cm	$l_1 - \Delta l$ cm	Area sq. cm	Stress psi	Strain %
σ_3 = 15 psi				1622.85		19.90	81.55		
at 15 sec			- 2.465	1620.38					
at 5 min			- 4.205	1618.64					
628	20	608	- 4.788	1618.06	0.110	19.79	81.76	48.0	0.553
1257	30	1227	- 3.516	1619.33	0.077	19.71	82.16	96.3	0.940
1885	46	1839	- 0.313	1622.54	0.097	19.62	82.70	143.4	1.427
2263	59	2204	+ 4.156	1627.01	0.089	19.53	83.31	170.6	1.874
2514	74	2440	+10.101	1632.95	0.094	19.43	84.04	187.2	2.347
2765	87	2678	+20.403	1643.25	0.135	19.30	85.14	202.9	3.025
2891	95	2796	+36.983	1659.83	0.199	19.10	86.90	207.5	4.205
3016	101	2915	+61.974	1684.82	0.300	18.80	89.62	209.8	5.533

TABLE 17

Typical Volume Change, Void Ratio and Percent Voids Computations

Type of Test: "Q"
Specimen: Q-12, 5%, 400 psi

Date: 3/23/60
By: JS

Volume of Asphalt = 166.36 cc
Volume of Aggregate = 1440.87 cc

Stress psi	ΔVol. cc	ΔVol. - V_{sv}* cc	Specimen Vol., cc	Vol. Air cc	% Change in Vol.	Voids %	Void Ratio
			1622.85	15.62		0.96	0.1263
0		- 2.465	1620.38	13.15		0.81	0.1246
15.0	- 4.205	- 1.740	1618.64	11.42	-0.107	0.71	0.1234
63.0	- 4.788	- 2.323	1618.06	10.83	-0.143	0.67	0.1230
111.3	- 3.516	- 1.051	1619.33	12.10	-0.065	0.75	0.1238
158.4	- 0.313	+ 2.152	1622.54	15.31	+0.133	0.94	0.1261
185.6	+ 4.516	+ 6.621	1627.01	19.78	+0.409	1.22	0.1292
202.4	+10.101	+12.566	1632.95	25.72	+0.775	1.57	0.1333
217.9	+20.403	+22.868	1643.25	36.02	+1.411	2.19	0.1405
222.5	+36.983	+39.448	1659.83	52.60	+2.434	3.17	0.1520
224.8	+61.974	+64.439	1684.82	77.59	+3.977	4.60	0.1693

* Assumes the specimen volume change represented by the 15 sec. reading is due to surface voids only

TABLE 18

Typical Computations of D_r and Sine ϕ_1

The following computations are based on data from a "Q" test at a confining pressure of 30 psi on a specimen prepared at 4 percent asphalt content and with a 400 psi foot pressure compaction. The volume of the specimen after the application of the confining pressure was 1635.18 cc.

D_r = Deformation Ratio = ratio of lateral strain to axial strain.

Computations of D_r

$\sigma_1 - \sigma_3$ psi	ΔV cc	ϵ_v %	ϵ_a %	$\epsilon_v - \epsilon_a$ %	$2\,\epsilon_a$ %	D_r
48.31	- 0.888	-0.054	-0.228	+ 0.174	- 0.556	0.31
97.33	- 1.207	-0.074	-0.352	+ 0.278	- 0.704	0.39
145.70	- 1.027	-0.063	-0.471	+ 0.408	- 0.942	0.43
192.90	- 0.107	-0.006	-0.600	+ 0.594	- 1.200	0.49
239.36	+ 1.779	+0.109	-0.778	+ 0.887	- 1.556	0.57
275.41	+ 4.865	+0.297	-1.031	+ 1.328	- 2.062	0.64
309.86	+12.309	+0.753	-1.482	+ 2.235	- 2.964	0.75
338.08	+46.496	+2.843	-3.401	+ 6.244	- 6.802	0.92
327.27	+86.084	+5.264	-5.434	+10.698	-10.864	0.98

ϕ_1= angle of shearing resistance

Computations of Sine ϕ_1

σ_1,psi	Z*,psi	σ_1-Z,psi	F	(F-1)	(F+1)	Sine ϕ_1
269.36	194.5	74.86	2.495	1.495	3.495	0.428
305.41		110.91	3.697	2.697	4.697	0.574
339.86		145.36	4.845	3.845	5.845	0.658
368.08		173.58	5.786	4.786	6.786	0.705
357.07		162.57	5.419	4.419	6.419	0.688

* $Z = 2c \tan (45 + \phi/2)$

APPENDIX C

TABLE 19

Typical Density Test Results

Design Asphalt Content, % by Weight of Mixture	Density, pcf					
	2.8		4.0		5.0	
	Measured	Displaced	Measured	Displaced	Measured	Displaced
Foot Pressure, 150 psi						
Entire Specimen	143.146	146.453	146.640	149.136	149.698	151.320
Top	142.147	145.642	145.018	148.574	148.824	151.320
Middle	142.958	144.830	145.954	148.886	149.011	151.195
Bottom	142.022	146.952	144.893	149.356	149.356	151.819
Maximum % Variation from Average	+0.4,-0.2	+0.8,-0.7	+0.5,-0.3	+0.3,-0.3	+0.3,-0.2	+0.3,-0.2
Foot Pressure, 250 psi						
Entire Specimen	145.018	147.576	147.451	150.509	150.821	152.880
Top	144.394	146.515	147.389	150.010		152.914
Middle	145.267	147.451	148.262	150.134		152.568
Bottom	144.643	149.198	147.014	150.758		153.317
Maximum % Variation from Average	+0.3,-0.3	+1.0,-0.8	+0.4,-0.4	+0.3,-0.2		+0.4,-0.3
Foot Pressure, 400 psi						
Entire Specimen	147.326	149.448	149.885	152.568	153.067	154.066
Top	145.766	148.637	148.949	150.883		153.629
Middle	147.264	149.573	150.883	152.693		154.752
Bottom	145.576	151.133	150.946	153.691		154.315
Maximum % Variation from Average	+0.5,-0.7	+0.9,-0.8	+0.5,-0.9	+0.8,-1.0		+0.3,-0.4

TABLE 20

Typical Asphalt Extraction Test Results

Foot Pressure, 150 psi

	Asphalt Content by Extraction % by Weight of Mixture		
Design Asphalt Content % by Weight of Mixture	2.8	4.0	5.0
Top	2.9	4.0	4.8
Middle	2.7	3.9	4.9
Bottom	2.7	4.1	5.0
Foot Pressure, 250 psi			
Top	2.7	4.0	4.4
Middle	2.7	3.8	4.5
Bottom	2.9	3.8	4.7
Foot Pressure, 400 psi			
Top	2.8	3.9	4.7
Middle	2.6	3.9	5.2
Bottom	2.5	3.8	4.9

APPENDIX D

TABLE 21

Typical Temperature Cycles

Controlled Temperature Room

Date: 2/14/60

Time	Temperature Reading, °C		
	A(1)	B(2)	C(3)
1:10	25.1	25.3	25.1
	24.2	24.5	24.5
	24.1	24.5	24.4
	24.7	25.0	25.0
	25.3	25.5	25.4
1:15	25.8	26.0	25.8
	25.8	25.9	25.8
	24.6	24.8	25.0
	24.0	24.4	24.6
	24.5	24.8	24.9
1:20	25.1	25.4	25.3
	25.7	26.0	25.8
	25.9	26.1	26.0
	24.9	25.1	25.2
	24.1	24.5	24.7
1:25	24.5	24.8	25.0
	25.2	25.4	25.4
	25.7	26.0	25.9
	26.0	26.2	26.0
	25.9	25.2	25.3
1:30	24.2	24.5	24.9
	24.4	24.8	25.0
	25.1	25.4	25.4
	25.7	26.0	25.8
	25.9	26.1	26.0
1:35	24.9	25.1	25.2
	24.2	24.5	24.6
	24.5	24.8	24.9
	25.1	25.4	25.3
	25.6	26.0	25.8
1:40	25.9	26.2	26.0

Average Temperature, °C at Location A = 25.05
at Location B = 25.30
at Location C = 25.30

Note: Temperature readings of the water in the water reservoir tank did not vary 0.1°C from 25.0°C during the period of observation.

(1) located at the sensing element of the temperature control
(2) located at the position of the triaxial cell
(3) located at the volume measuring device

TABLE 22

Volume Change Due to Piston

In the performance of a test, a 3/4-inch diameter hardened steel piston is used to apply load to the specimen. As this piston moves into the cell upon deformation of the specimen, the volume of the cell liquid is changed. To compensate for this volume change, a correction must be applied to the observed volume change readings.

The piston correction is computed on the basis of the volume of liquid displaced by the 3/4-inch (1.9025 cm) diameter piston moving a distance of 0.001 cm. Tables were prepared that permitted direct evaluation of the piston displacement resulting from any given extensometer change. The portion of the table for movements of 0.000 to 0.099 cm was used most frequently and is the portion selected as illustrative of this computation and the method of establishing the piston displacement correction.

Volume Change, cc

Deformation cm	0.000	0.001	0.002	0.003	0.004	0.005	0.006	0.007	0.008	0.009
0.00	-	0.003	0.006	0.008	0.011	0.014	0.017	0.020	0.023	0.025
0.01	0.028	0.031	0.034	0.037	0.040	0.043	0.045	0.048	0.051	0.054
0.02	0.057	0.060	0.062	0.065	0.068	0.071	0.074	0.077	0.080	0.082
0.03	0.085	0.088	0.091	0.094	0.097	0.099	0.102	0.105	0.108	0.111
0.04	0.114	0.116	0.119	0.122	0.125	0.128	0.131	0.134	0.136	0.139
0.05	0.142	0.145	0.148	0.151	0.153	0.156	0.159	0.162	0.165	0.168
0.06	0.170	0.173	0.176	0.179	0.182	0.185	0.188	0.190	0.193	0.196
0.07	0.199	0.202	0.205	0.207	0.210	0.213	0.216	0.219	0.222	0.225
0.08	0.227	0.230	0.233	0.236	0.239	0.242	0.244	0.247	0.250	0.253
0.09	0.256	0.259	0.262	0.264	0.267	0.270	0.273	0.276	0.279	0.281

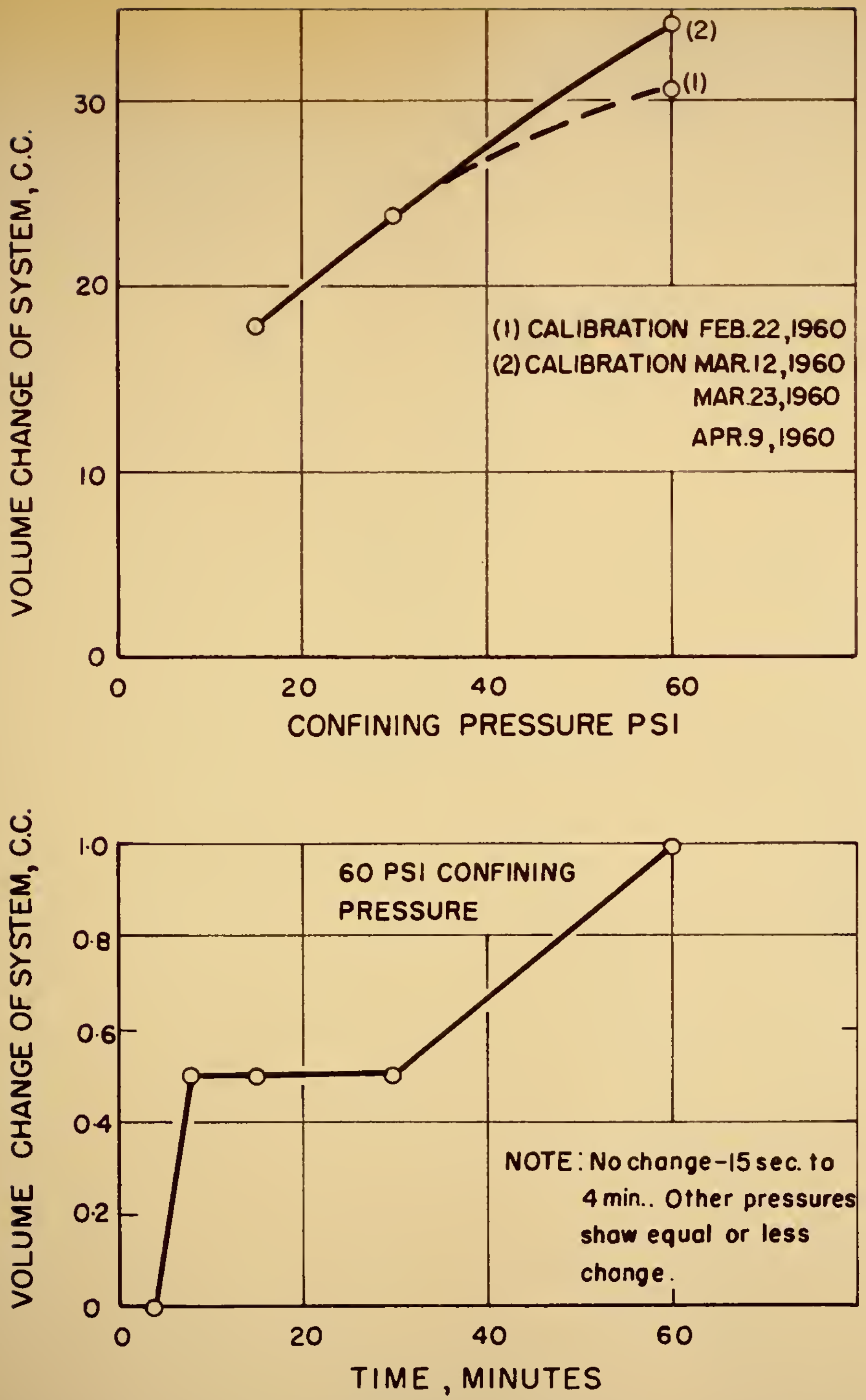


FIGURE 35. CALIBRATION CURVES FOR VOLUME MEASURING DEVICE AND TRIAXIAL CELL

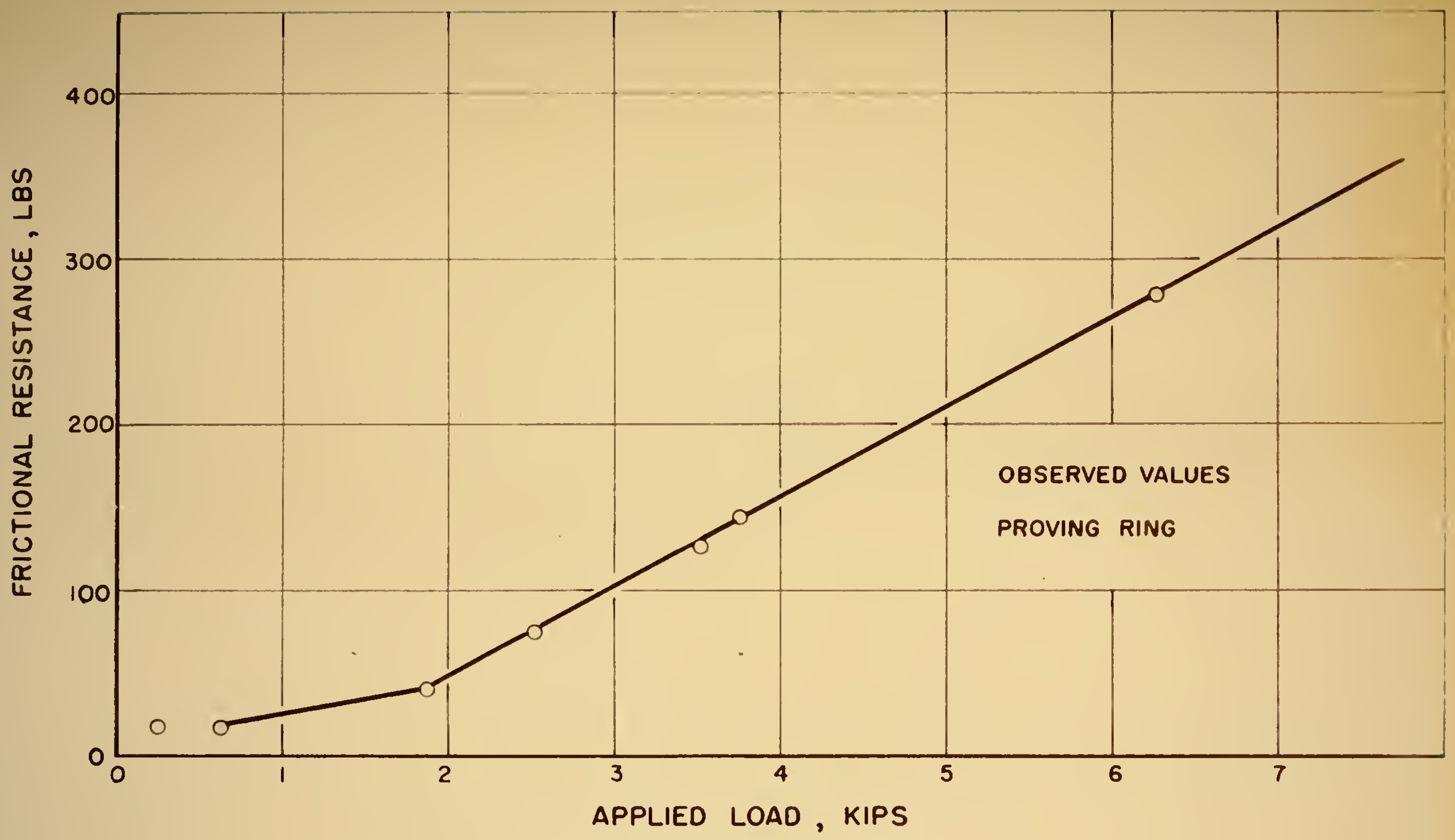
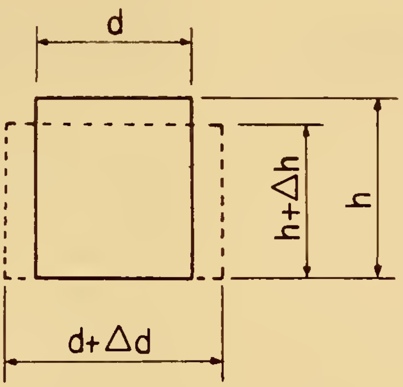


FIGURE 36. FRICTIONAL RESISTANCE CALIBRATION CURVE

APPENDIX E

Deformation Ratio and Angle of Shearing Resistance

1. Consider a right cylinder of initial height h and diameter d and having these dimensions changed by an incremental amount under load.



2. The initial volume of the cylinder = V = $\frac{\pi d^2}{4}$ (h)

3. Volume after compression is $V + \Delta V = \frac{\pi}{4} (d + \Delta d)^2 (h + \Delta h)$

4. For small values of Δd and Δh

   $V + \Delta V = V (1 + \frac{2 \Delta d}{d} + \frac{\Delta h}{h})$

5. If $\frac{\Delta d}{d}$ is defined as lateral strain (ϵ_1) and $\frac{\Delta h}{h}$ as axial strain (ϵ_a) then $\frac{\Delta V}{V} = 2 \epsilon_1 + \epsilon_a$

6. Volumetric strain is defined as $\epsilon_v = \frac{\Delta V}{V} = 2 \epsilon_1 + \epsilon_a$

7. The Deformation Ratio (D_r) is analogous to Poisson's Ratio and is defined in a similar manner as the ratio of lateral strain to axial strain. Therefore $D_r = - \epsilon_1 / \epsilon_a$

8. Substituting from Step 6 and using the definition of Step 7

$$D_r = -\frac{\epsilon_v - \epsilon_a}{2\,\epsilon_a}$$

9. Friction component of stress

$$\tan^2(45 + \phi/2) = \frac{\sigma_1 - 2c\tan(45 + \phi/2)}{\sigma_3} = (1 + \sin\phi)/(1 - \sin\phi)$$

10. Defining $\frac{(1 + \sin\phi)}{(1 - \sin\phi)} = F$ then $\sin\phi = \frac{(F - 1)}{(F + 1)}$

Relationship Between Percent Voids and Void Ratio

Assumed to be known:

Percent Voids $= v = (V_a/V) \times 100 = \dfrac{\text{Volume of Air Voids}}{\text{Volume of Specimen}} \times 100$

Asphalt Content, % by weight of mixture = C

$= \dfrac{\text{total weight of asphalt}}{\text{weight of specimen}} \times 100$

Bulk Density of Specimen $= \gamma = \dfrac{\text{weight of specimen}}{\text{volume of specimen}}$

Percent Asphalt Absorbed = A

$= \dfrac{\text{weight of asphalt absorbed by aggregate}}{\text{weight of aggregate}} \times 100$

Specific Gravity of Asphalt = G

Unit Weight of Water $= \gamma_w$

Find: Void Ratio in Terms of Known Quantities

By Definition: Void Ratio $= e = \dfrac{\text{Vol.Air + Vol.Asphalt Filling Voids}}{\text{Vol. Aggregate}}$

1. considering a unit volume of specimen

   volume of air $= V_a = v/100$

2. total weight of asphalt $= (C/100)\ \gamma$

3. weight of aggregate in specimen $= \gamma - (C/100)\ \gamma$

4. weight of asphalt absorbed $= (\gamma - C\gamma/100)(A/100)$

5. weight of asphalt effective in filling voids

   $= (C\gamma/100)(1 + A/100) - (A\gamma/100)$

6. volume of asphalt filling voids $= V_b$

   $= \dfrac{(C\gamma/100)(1 + A/100) - (A\gamma/100)}{G\ \gamma_w}$

7. volume of aggregate $= 1 - V_a - V_b$

8. void ratio = $e = (V_a + V_b)/(1 - V_a - V_b)$

   a. note that V_a and V_b are established from steps 1 and 6 respectively.

VITA

VITA

James Hamilton Schaub was born on January 27, 1925, in Moundsville, West Virginia. He attended the grammar and high schools of that city and completed his high school education at the Capitol Page School, Washington, D. C., in June, 1942.

Mr. Schaub attended George Washington University from June, 1942, until March, 1943, as a part-time student while employed in the Post Office of the Senate Office Building.

Mr. Schaub entered the United States Army in April, 1943. Following basic training, he attended Virginia Polytechnic Institute under the ASTP program. In April, 1944, he was assigned to the 334th Infantry Regiment and completed his World War II military service, including combat and occupational duties in Europe, with that unit.

Upon completion of military service, Mr. Schaub re-entered Virginia Polytechnic Institute and received the Bachelor of Science in Civil Engineering degree in June, 1948. He attended Harvard University the following year and was awarded the Master of Science degree in June, 1949.

Mr. Schaub's experience includes three years employment as Chief Soils Engineer for the Oregon Highway Department and three years as Laboratory Director for Palmer & Baker, Engineers, Inc. of Mobile, Alabama. The former employment was interrupted by active duty as a first lieutenant with the Corps of Engineers in Korea from September, 1950 until August, 1951.

In 1955, Mr. Schaub was appointed Assistant Professor of Civil Engineering at Virginia Polytechnic Institute. In this position he was responsible for graduate and undergraduate instruction in soil mechanics and transportation engineering.

Mr. Schaub continued his graduate school education by attending Purdue University during the summers of 1956, 1957 and 1958. In September, 1958, he was granted leave from Virginia Polytechnic Institute and employed as an instructor in the School of Civil Engineering of Purdue University.

Mr. Schaub was married in June, 1948, to the former Malinda Bailey of Radford, Virginia.

Mr. Schaub is a member of the Society of Sigma Xi (Associate), American Society of Civil Engineers (Member), and the Highway Research Board (Associate). He is a registered professional engineer in Virginia. He is a member of Phi Kappa Phi, Tau Beta Pi, Chi Epsilon, and Kappa Sigma fraternities.

His publications are:

Schaub, J. H., "Soils Exploration and Design Considerations for the Greater New Orleans Expressway. Part I - Exploration and Testing," Proceedings, Highway Research Board, Vol. 35, 1956.

Goetz, W. H. and J. H. Schaub, "Triaxial Testing of Bituminous Mixtures," Special Technical Publication 252, American Society for Testing Materials, 1959.

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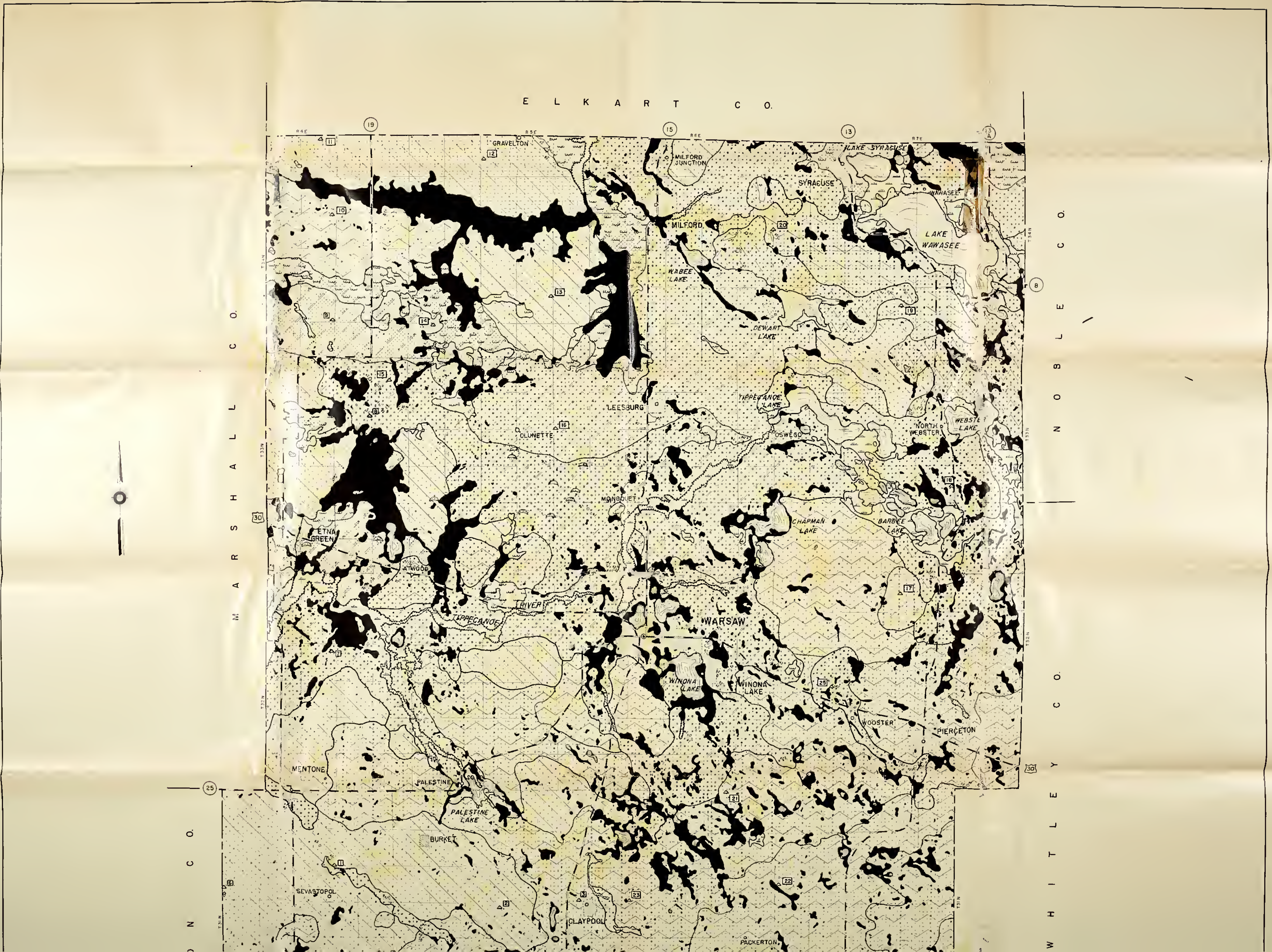
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SYRACUSE
WAWASEE
LAKE WAWASEE
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WEBSTER LAKE
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CHAPMAN LAKE
BARBEE LAKE
ETNA GREEN
ATWOOD
TIPPECANOE
RIVER
WARSAW
WINONA LAKE
WINONA LAKE
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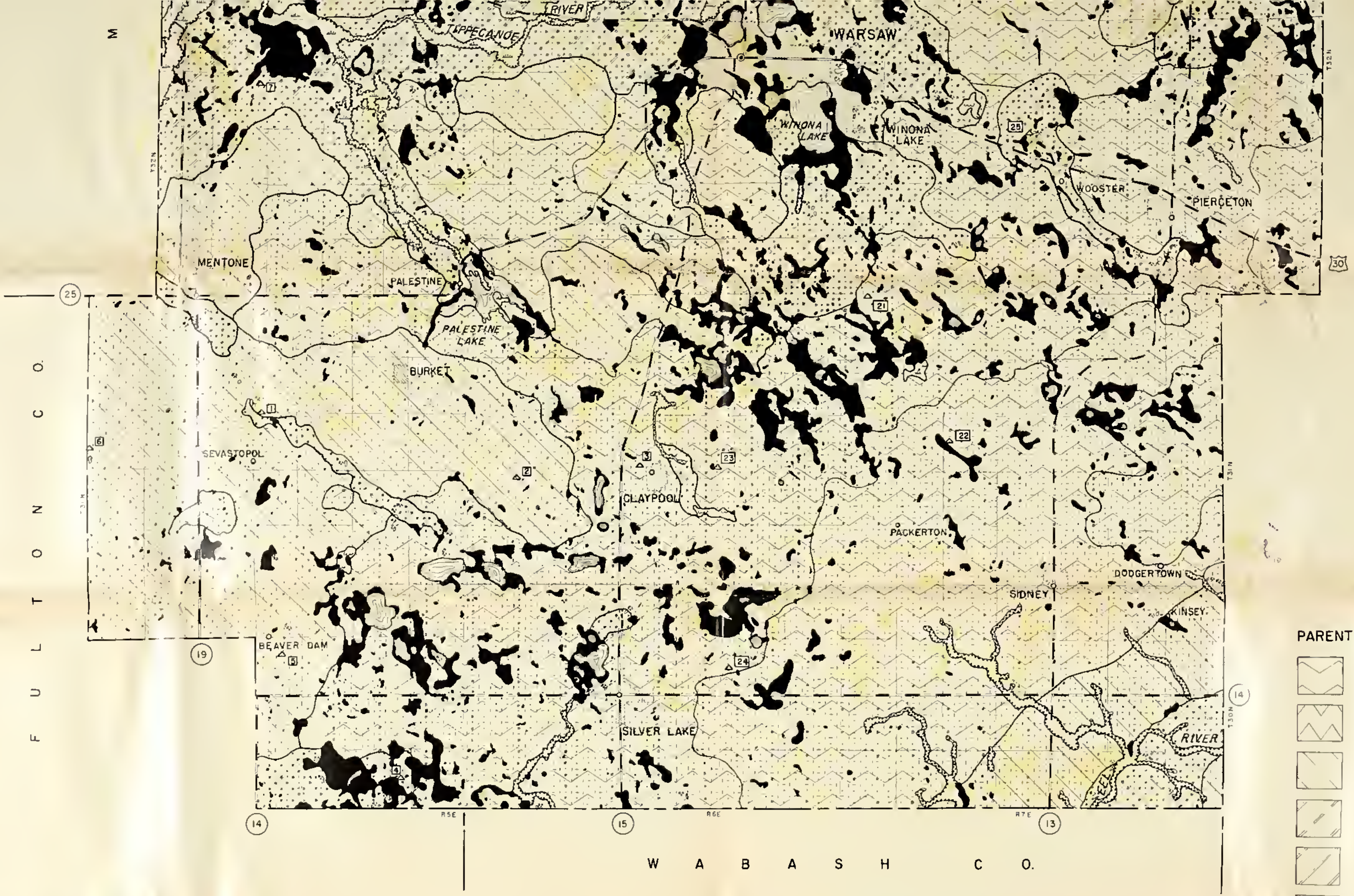

TEXTURAL SYMBOLS (SUPERIMPOSED ON PARENT MATERIAL SYMBOLS TO SHOW RELATIVE COMPOSITION)

- GRAVEL
- SAND
- SILT
- CLAY

ENGINEERING SOILS MAP
KOSCIUSKO COUNTY
INDIANA

PREPARED FROM
1951 AAA AERIAL PHOTOGRAPHS
BY
JOINT HIGHWAY RESEARCH PROJECT

PURDUE UNIVERSITY

1960

SCALE OF MILES
1 3/4 1/2 1/4 0 1 2 3 4

LEGEND

PARENT MATERIALS (GROUPED ACCORDING TO LAND FORM AND ORIGIN)

- MORAIN
- KETTLE KAME MORAIN
- GROUND MORAIN
- OUTWASH PLAIN
- OUTWASH PLAIN TRANSITION
- VALLEY TRAIN, TERRACE
- ESKER
- KAME
- ALLUVIAL PLAIN, RECENT
- SAND DUNES

MISCELLANEOUS

- PEAT AND MUCK
- HIGHLY ORGANIC TOP SOIL
- MARSH
- GRAVEL PIT
- CLAY PIT
- LAKE AND POND

PREPARED UNDER THE SUPERVISION OF P. T. YEH AND J. NARAIN

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